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Vol. 26—No. 10 June 1955 (Proceedings Vol. 51)

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DON'T FORGET—

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Structural Model Studies of Concrete Slab Foundations*

By C. DESMOND PENGELLEY,† E. J. DOWER,† and M. M. LEMCOE‡

SYNOPSIS

The theory of dimensional analysis was applied to the design of reinforced concrete structural models of small home foundation slabs. Methods of fabricating satisfactory models were developed. Static stiffness measurements are reported on six different designs of the same cost and size. Two full-scale foundations were tested corresponding to a pair of the model designs, and model and full-scale results are compared.

INTRODUCTION

Throughout the southwestern states, particularly Texas, Arkansas, Colorado, and New Mexico, unstable soil conditions are prevalent which constitute a severe problem with regard to the foundations of permanent buildings. The reasons for this instability are not thoroughly understood, and some study is being given to the problem by various agencies.[1-4] Suffice it to say that samples of soil exhibit a marked tendency to swell as the moisture content increases and to shrink upon drying.

When solid rock is available at a reasonable depth beneath the surface, it may be used to support the foundation; however in many places this is not possible, and the foundation must be laid on the soil itself.

During the past decade monolithic slab foundations have been developed which virtually "float" on top of the soil. When the soil under the foundation heaves or settles in an irregular manner, it subjects the slab to bending and torsion in much the same way as a ship's hull is stressed by the waves of the ocean. If the slab is stiff enough, it will maintain its shape and protect the structure of the house from warpage and cracks. However, due to the great weights involved (a typical slab usually weighs several times as much as the house itself), slab stiffness becomes the most important single parameter in the design of this type foundation.

Many different designs have been tried with varying degrees of success. One in particular has been approved for use in homes insured by the FHA, and it has become almost a standard. This slab incorporates a perimeter

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beam with secondary internal beams which are covered with a relatively thin floor. For convenience, this design will be referred to as the "conventional slab," and it has been used as a standard of comparison during the work reported herewith.

Late in 1950 a program was established at the Southwest Research Institute to develop an alternate slab design which would result in a foundation having greater stiffness than was possible with the conventional slab but at no increased cost.

Due to the inherent difficulties and high cost of building and testing full-scale foundations with controlled loading conditions and realistic measurements, it was decided to make use of the theories of dimensional analysis and carry out the program on a model scale in the laboratory where uniform conditions could be achieved.

It can be seen that there are an infinite number of possible loading distributions which may arise under a slab on unstable soil. To reduce the program to a practical scope, three arbitrary loading conditions were devised which would represent extremes and which could be combined to approximate any condition that might be experienced in practice. These are described under "Model Studies," p. 966.

Coincident with the Southwest Research Institute program, the Portland Cement Assn. was also making elevation measurements on the foundations of several newly-built homes in the San Antonio area. Of the five houses built, only one showed any settlement below the initial elevation. That one was constructed on a one year old poorly compacted fill of from 4 to 7 ft deep. After a few months it, too, started to rise. All five houses have slowly risen almost without regard to weather conditions. The maximum movement to date is about $7\frac{5}{8}$ in. The average movement is between $2\frac{1}{2}$ to 3 in. This clearly indicates an instability due to varying differential reactive pressure and shows that the problem is not one relating to an overloaded foundation with excessive bearing pressures.

The model techniques on which this program has been based have been subsequently applied to other reinforced concrete structures including prestressed structures; however the present report is limited to the study of foundation slabs.

DIMENSIONAL THEORY

The theory of dimensional analysis provides procedures by which data obtained during a laboratory test may be applied to a similar apparatus built to a different scale. The procedure consists of two steps:

(1) Determination of scale ratios for all important parameters.
(2) Development of a suitable method of model construction.

Doherty and Keller[5] have summarized the theory of dimensional analysis by means of which the scale ratios may be established for any given type of apparatus. In applying this theory to the consideration of strength and stiffness of a statically loaded structure, it is first necessary to make certain

that the model is a true scaled replica of the prototype. In the case at hand this means that all dimensions, including concrete shape as well as steel sizes and location, must be in exact proportion between model and full scale. It also means that load distribution must be the same for model and full scale so that the laws of similarity can be applied. In addition, materials having similar structural properties should be used; that is to say, Poisson's ratio and the stress-strain curves for the materials of model and full scale should be the same.

There are four important dimensional quantities which must now be considered to apply model data to the prediction of full-scale performance. These are force, F, length, L, density, σ, and stress, f. These may be combined to yield two nondimensional ratios:

$$\frac{F}{fL^2} \text{ and } \frac{L\sigma}{f}$$

The first of these establishes correspondence between applied load, F, and stress, f, for different sizes, L, while the second pertains to the relationship between dead weight loading—due to density, σ—and stress as a function of size.

In the model program described here, steel and concrete of substantially the same physical properties as the full-scale materials were used. For this reason corresponding stresses were the same between model and full scale. The following relationships were, therefore, applicable between model and full scale (where subscripts M and F refer to model and full scale, respectively):

$$f_M = f_F \tag{1}$$

$$\frac{F_M}{F_F} = \left(\frac{L_M}{L_F}\right)^2 \tag{2}$$

$$\frac{\sigma_M}{\sigma_F} = \frac{L_F}{L_M} \tag{3}$$

The requirements of Eq. (2) may be described in the following manner. The experimenter may establish any convenient linear scale factor (L_M/L_F), say 1/12, to determine the model size. Having done this, he may obtain the static forces to be used on the model by reducing the equivalent full-scale forces in proportion to the square of the scale factor.

Eq. (3), however, introduces a problem. It requires that the density of all model materials should be changed inversely with respect to the linear scale factor. If the same materials are to be used for model and full scale, as specified above, the requirement of Eq. (3) cannot be met. This difficulty may be resolved for static strength and deflection tests when it is understood that Eq. (2) sets up the requirement that external forces shall be similar while Eq. (3) refers specifically to dead weight loads. Thus, Eq. (3) may be satisfied indirectly by applying arbitrary external forces to the model, which are distributed in such a manner as to provide a loading that is substantially the same as that which would have been set up by gravity if the density of the model materials had really been increased as required.

It has already been pointed out that model materials must have the same properties as full-scale materials. In the case at hand, consideration must be given to (1) type of steel, (2) suitable roughening of steel surface, (3) concrete stress-strain data, and (4) aggregate size. With regard to the type of steel, it is probably sufficient to verify that the yield point and ultimate stress of the reinforcing steel are the same for model and full scale without checking the stress-strain characteristics over the entire range. The model wire should be deformed so as to provide a roughened surface that is similar to full-scale reinforcing steel. Coarse aggregate should be scaled down directly in proportion to the scale of the model; this is essential to permit passage between the reinforcing wires. It has been deemed permissible to ignore the scale factor on the cement dust because the particle size is negligible on both model and full scale. By somewhat the same reasoning, the use of fine clean sand, but without actual scale control of the grain size, is justified for the models. The model mix and water proportions should be the same as full scale, and ultimate strength and modulus of elasticity should be checked.

The foregoing requirements assure a model structure which is an exact scale replica of the prototype. Static tests may be made on such a model and the results applied to the full scale as follows:

(1) Corresponding total full-scale loading will be in proportion to the square of the scale.

(2) The full-scale distribution of loading must be the same as for the model.

(3) Full-scale deflections will be in direct proportion to the scale.

TECHNIQUE OF MODEL CONSTRUCTION

Model design specifications have been set forth above, but little has been said with regard to the fabrication techniques that may be used to achieve these requirements. A successful model must have the following characteristics:

(1) Must be cheap compared to full-scale cost.

(2) Dimensional tolerances must be held to scale.

(3) Models must be reproducible with a high percentage of success.

Mild steel baling wire was used for reinforcing. It was passed through a knurling machine to produce deformities similar to the full scale, thus insuring an adequate bond with the concrete. Tests indicated that the bond strength developed on a length of 30 diameters was sufficient to exceed the ultimate strength of the wire. It was straightened by hanging a weight on it and then heating it electrically until yielding occurred. The current was supplied by a welding machine. Exact values of tension and current varied for each wire size, but with a little experimentation, suitable conditions were found which would permit straightening without affecting the yield point and ultimate strength for several specimens whose yield point and ultimate strength were 40,000–45,000 psi and 55,000–70,000 psi, respectively. Wire diameters were chosen in proportion to full-scale sizes, and stirrups

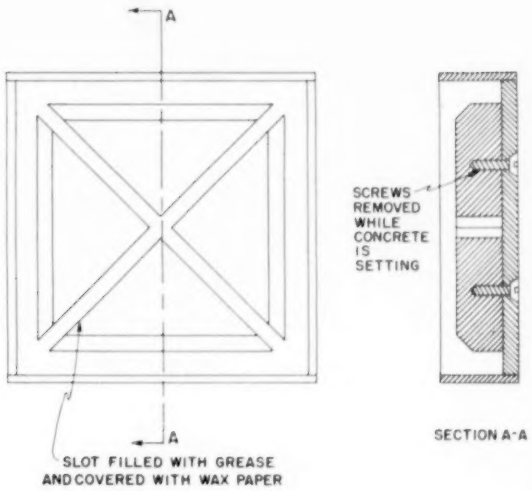


Fig. 1—Method of making internal cores

and ties were provided in corresponding places. The tying was done with 26-gage Nichrome wire handled with surgical forceps.

The coarse aggregate was scaled down by suitable screening while clean fine sand and good cement were used without control of the grain size.

Slabs were cast in molds of well-seasoned and smoothly-finished maple sprayed with three coats of clear nitrate airplane dope. All molds were put together with screws so that they could be removed piece by piece from the model. Internal cores were made as shown in Fig. 1. The core itself was made in pie-shaped sections with a gap of about 1/16 in. between each. Each section was secured in its proper place with a screw through the bottom of the mold. The gaps were filled with heavy grease which was wiped off clean with the surface of the core, and the slot was then covered with a strip of waxed paper.

After the steel had been cut and tied, it was placed in the mold and the concrete placed and vibrated in conventional fashion. A few hours after casting, when the concrete had started to set, the indexing screws which held the core sections in place were removed. The sections were then free to slide toward each other as the concrete tended to shrink.

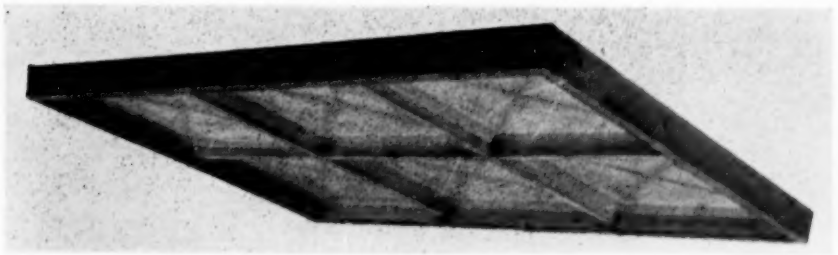


Fig. 2—Underside of conventional slab

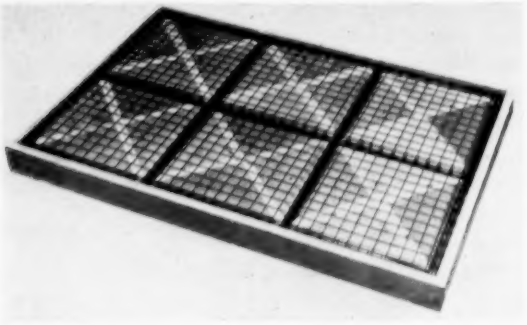


Fig. 3—Steel for conventional slab

This procedure was the result of much experimentation and has since been used successfully with many different concrete models. The type of wood finish is essential to prevent sticking, and the technique of making collapsible cores is needed to prevent severe shrinkage cracks as well as to make it possible to remove the core after the concrete is set.

The concrete was mixed in the ratio of one part cement, two parts sand, and four parts gravel and a water-cement ratio of 0.665.

MODEL STUDIES

Description of models

Each model was 24 x 36 in. A model of a conventional slab was made first as a 1/12-scale replica of a 24 x 36 ft small home foundation as specified for approval for insurance by the FHA in the San Antonio area (Fig. 2 and 3).

Five experimental designs were then made:

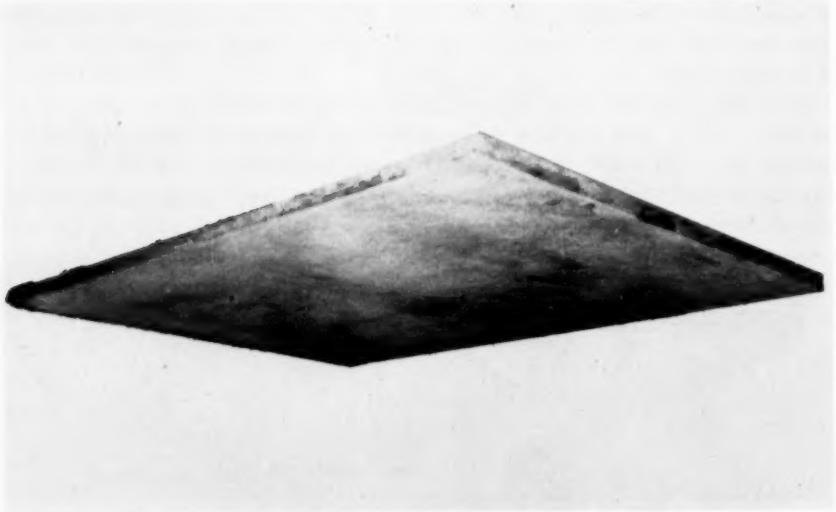


Fig. 4—Underside of flat slab

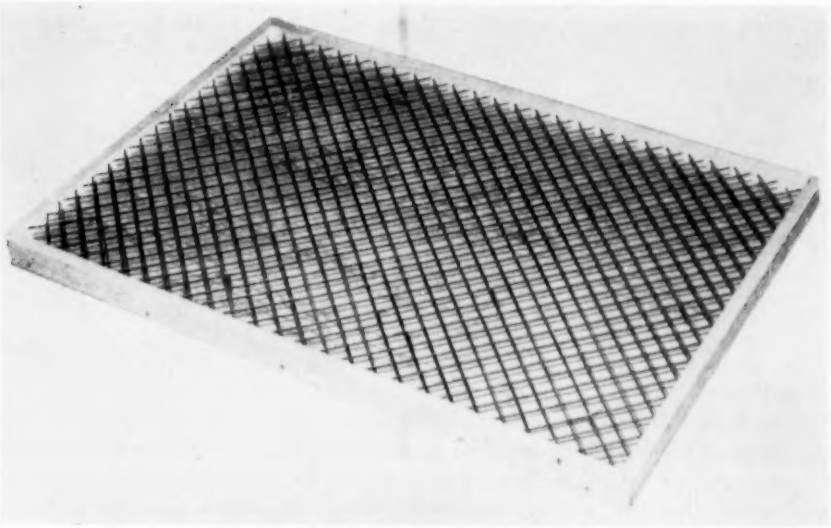


Fig. 5—Steel for flat slab (diagonally reinforced)

(1) Flat slab with reinforcing mat at top and bottom with rods running parallel to the sides of the slab (Fig. 4).

(2) Flat slab same as (1) above except reinforcing rods running at 45 deg to sides of the slab (Fig. 4 and 5).

(3) Inverted rectangular pyramid with reinforcing mat near each surface with rods running parallel to sides of slab (Fig. 6).

(4) Same as inverted pyramid above except reinforcing rods at 45 deg to sides of slab (Fig. 6 and 7).

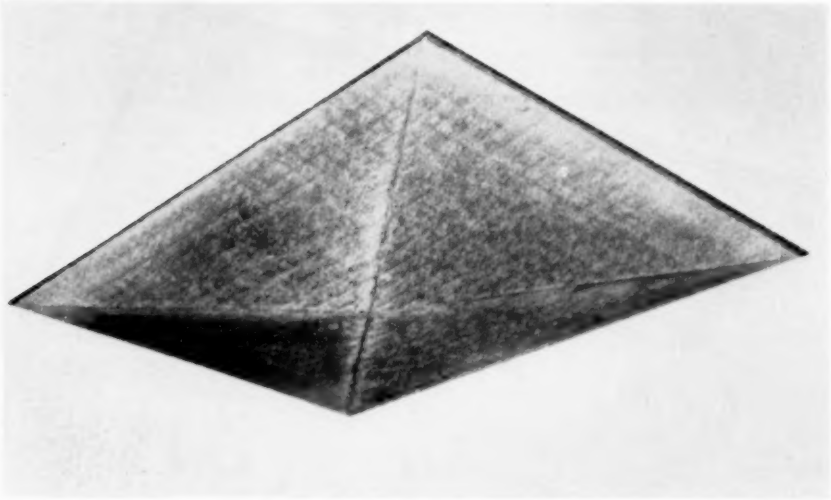


Fig. 6—Underside of pyramidal slab

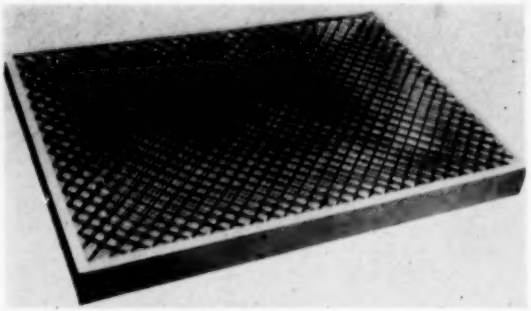


Fig. 7—Steel for pyramidal slab (diagonally reinforced)

(5) X-keel slab which makes use of a perimeter beam and a tapered pair of diagonal beams which are deep at the center and taper toward the corners (Fig. 8 and 9).

In each of the experimental designs a careful estimate was made of the probable full-scale cost; in fact the amount of materials was chosen to make the total cost substantially the same as for the conventional slab. The cost breakdown shown in Table 1 represents the average of estimates made by several different men with experience in building and contracting and indicates that the designs represented foundations which would actually have been of equal cost within estimating accuracy.

Experimental procedure

There are an infinite number of ways in which the soil may shift and heave so as to support the foundation in an irregular fashion. To obtain comparative data between slabs of different design, it is essential to establish a finite number of typical but critical types of loading which may occur. Different

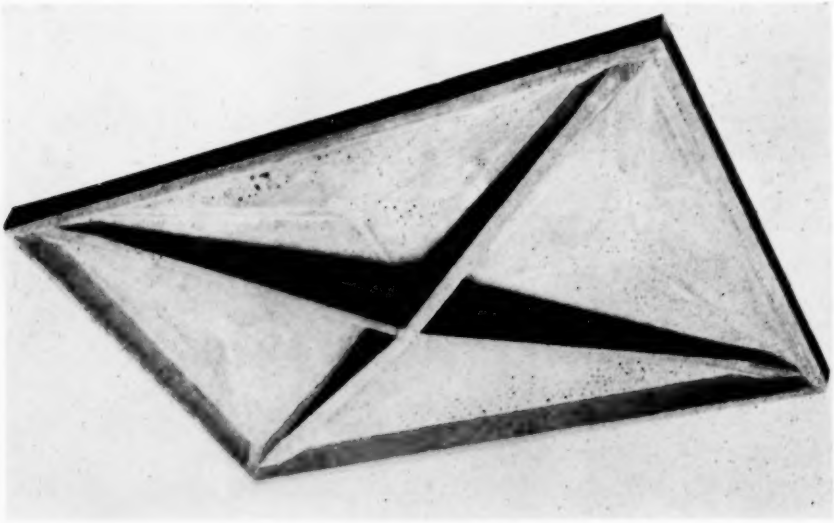


Fig. 8—Underside of X-keel slab

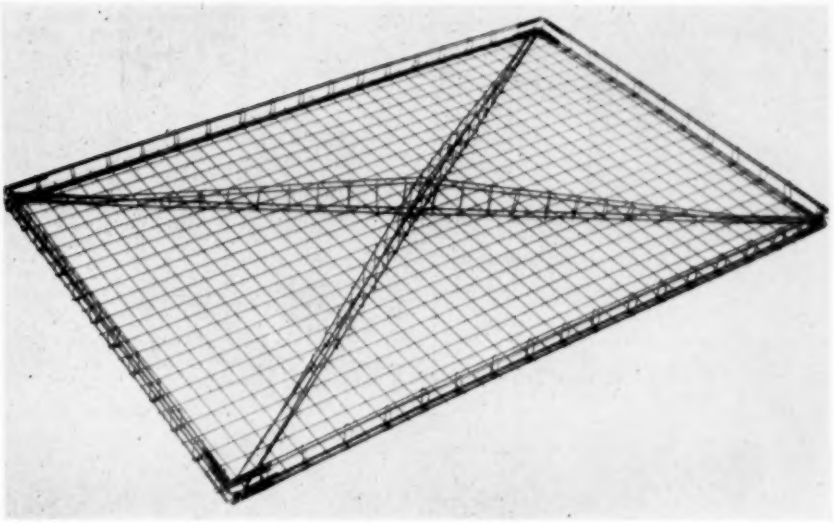


Fig. 9—Steel for X-keel slab

designs may then be tested comparatively for each of these loadings. Three such loading conditions were chosen as being representative of the extremes which may be encountered in practice. These involve the slab being supported from different areas as shown by shading in Fig. 10. These loading conditions have been described as "end," "center," and "corner," respectively.

For each loading condition the slab was supported on 2-in. thick pads of sponge rubber. The pads were cut to the appropriate shape as shown in Fig. 10, and due to their softness it may be assumed that they provided uniform pressure over their supporting area.

As pointed out following Eq. (3), it is necessary to apply an external load to make the model's dead weight deflection proportional to the corresponding full-scale dead weight deflection. Since the models were designed to be

TABLE 1—COMPARATIVE COSTS OF SLABS*

Item	Conventional		Flat slab		Pyramid		X-keel	
	Labor	Material	Labor	Material	Labor	Material	Labor	Material
Excavating	$0.029	$—	$0.019	$—	$0.088	$—	$0.029	$—
Gravel	0.020	0.030	0.016	0.044	0.016	0.044	0.020	0.030
Membrane	0.060	0.030	0.030	0.022	0.030	0.022	0.060	0.030
Forms	0.025	0.025	0.025	0.025	0.013	0.008	0.025	0.025
Steel	0.070	0.210	0.050	0.186	0.050	0.186	0.080	0.210
Concrete	0.110	0.300	0.110	0.388	0.110	0.388	0.110	0.337
Traveling	0.020	—	0.020	—	0.020	—	0.020	—
Stripping	0.011	—	0.011	—	0.011	—	0.011	—
Total	0.345	0.595	0.281	0.665	0.338	0.648	0.355	0.632
Labor and material	$0.940		$0.946		$0.986		$0.987	

*Full-scale designs 24 x 36 ft. Costs in dollars per sq ft.
Cost of concrete—$11.80 per cu yd.
Cost of gravel — 2.38 per cu yd.
Cost of steel — 0.071 per lb.

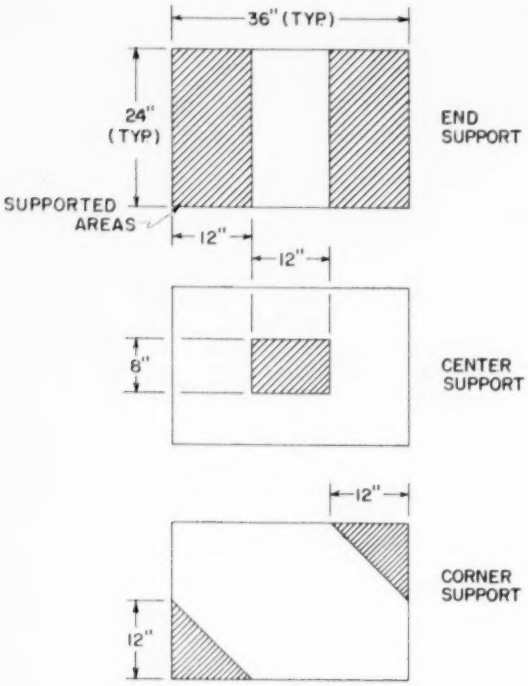


Fig. 10—Supporting areas used to apply different loading conditions

1/12 scale, each was loaded with lead shot to a total dead weight loading of 12 times its own weight. This provided model data which may be scaled up (Table 2) for comparison with deflections of a full-scale slab supported in the same fashion and carrying its own dead weight (Table 3).

Deflection readings were taken with dial gages at a number of points over the surface of each slab so that deflection contours were obtained. For simplicity in recording summarized data, a single deflection may be referred to in any given case. This deflection is obtained by passing a plane through three corners of the slab and then recording the maximum deflection of any point in the slab from this plane. Such deflections have been listed in Table 4.

TABLE 2—EQUIVALENT FACTORS BETWEEN MODEL AND FULL-SCALE SLABS

Factor	Model	Full scale
Scale factor	$L/12$	L
Shape	Rectangular	Rectangular
Size	24 x 36 in.	24 x 36 ft
Force, lb	$F/144$	F
Stress, psi	f	f
Pressure, lb per sq ft	p	p
Deflection, in.	$\Delta/12$	Δ

The model test set up is illustrated diagramatically in Fig. 11. The dial gages were mounted on a "floating frame" which was supported from the slab surface at three points. These three points constituted a zero plane of reference for plotting contours. In this way the frame itself was not subject to any strain, while over-all rotation and translation of the model were automatically eliminated from

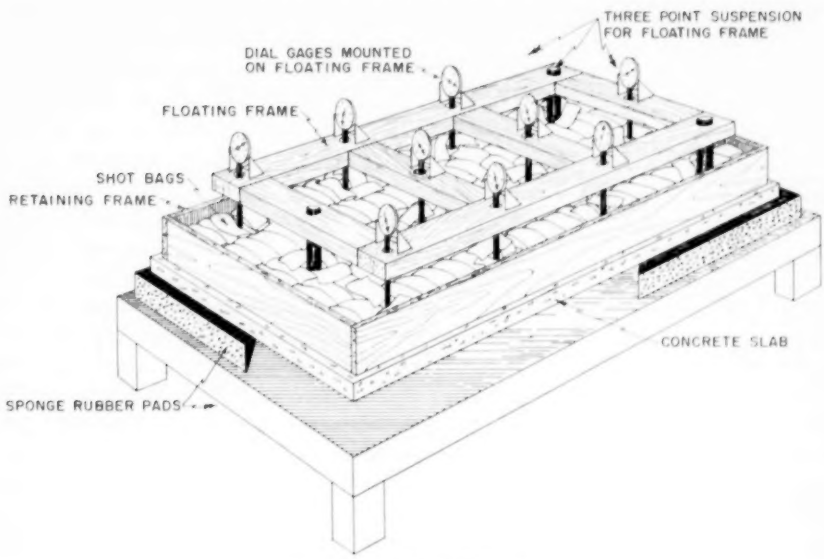


Fig. 11—Model loading jig

the readings. Extension rods were provided for the dial gages. These rods were positioned on the slab by suitable indicating pads and guided at their top ends by clearance holes in the floating frame. In this way the dial gages were protected from harm while loading and unloading the shot bags. It should be noted that friction between the shot bags and extension rods did not affect the readings since relative motion took place only between the rods and the floating frame. A light retaining frame which rested on the slab was provided to keep the shot bags in place.

Comparison of experimental and conventional slabs

In Table 4 it can be seen that the corner loading produces by far the greatest deflection in all cases. For this reason the corner loading condition only will be used for purposes of comparison.

Since the various slab designs had different weights (see Table 3), the deflection measurements listed in Table 4 do not truly represent comparative stiffness. For this reason Table 5 was prepared in which the slab weights are divided by corre-

TABLE 3—COMPARATIVE WEIGHTS OF SLABS

Design	Equivalent full scale weight, lb
Conventional	90,000
Flat	115,000
Pyramid	115,000
X-keel	99,500

TABLE 4—MODEL DEFLECTION MEASUREMENTS

Design	Deflection for type of loading, in.		
	End	Center	Corner
Conventional	0.037	0.015	0.257
Flat (rectangular)*	0.015	0.021	0.202
Flat (diagonal)†	0.019	0.018	0.122
Pyramid (rectangular)*	0.008	0.009	0.200
Pyramid (diagonal)†	0.012	0.011	0.093
X-keel	0.024	0.012	0.169

*Steel placed parallel to sides.
†Steel placed 45 deg to sides.

TABLE 5—COMPARATIVE STIFFNESS OF DIFFERENT DESIGNS FOR CORNER LOADING

Design	Relative stiffness, percent
Conventional	100
Flat (rectangular)*	163
Pyramid (rectangular)*	164
X-keel	168
Flat (diagonal)†	269
Pyramid (diagonal)†	354

*Steel placed parallel to sides.
†Steel placed 45 deg to sides.

sponding deflections due to corner loading, and the stiffnesses so obtained have then all been compared with that of the conventional design.

Another factor that must be discussed in comparing stiffness is the effective position of the supporting reaction. In the corner-type loadings each model slab was supported by a triangular piece of sponge rubber (see Fig. 11). Though the same size pads were used for each design, the configuration of the bottom of the slab probably had an effect on the supporting pressure distribution. In the two extreme cases, namely, the flat slab and the conventional beam-type slab, the former was supported by substantially a uniform pressure over the rubber triangle while the latter made contact primarily along the beams. The center of pressure of a uniformly loaded triangle is at one-third the height; however the center of pressure of a uniform loading along the two slant sides of a triangle is at one-half the height. This makes the effective diagonal beam length of the conventional slab longer than that of the flat slab. An approximate analysis has indicated that this had the effect of making the flat slab model appear to be relatively 25 percent too stiff. It is probable that soil beneath a foundation acts in the same way, however, and this apparent advantage enjoyed by the flat slab is undoubtedly quite justified in actual building practice. For this reason no allowance need be made for effective diagonal beam length when comparing the stiffness of different designs; however this must not be ignored when comparing model and full-scale results as shown later.

A study of Table 5 shows in particular the beneficial effect of a diagonal mat as compared with a rectangular mat. The corner loading stiffness of the flat slab was increased approximately 65 percent due to diagonal placement of the steel alone; this was obtained at the expense of a 21 percent reduction in end-loading stiffness. The effect was even greater for the pyramid design. Since deflections under corner loading were many times greater than any other loading condition, this represents a considerable net gain by using diagonal mats instead of rectangular.

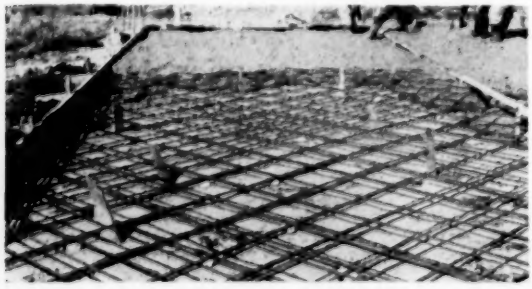


Fig. 12—Steel for full-scale flat slab with diagonal reinforcement

Table 5 also shows that the diagonal-mat flat slab was 2.69 times as stiff as the conventional slab under corner load while the same factor for the diagonally reinforced pyramid was 3.54. Though the diagonal-mat pyramid was the stiffest of all designs tested, it was concluded that the extreme simplicity of the flat slab would be most acceptable to builders and could be readily applied to foundations of irregular shape. For this reason the diagonal-mat flat slab was used for full-scale studies as described in the following section.

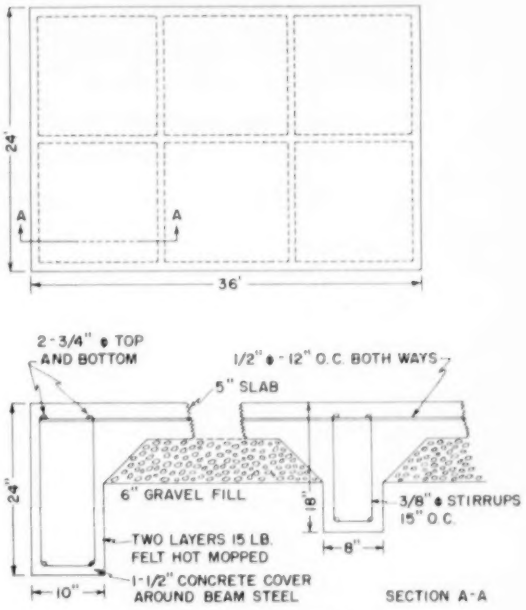


Fig. 13—Conventional slab

FULL-SCALE STUDIES

Description of experimental foundations

As a result of the model studies, the flat slab with diagonal reinforcing was chosen as the most promising over-all design after taking into account the attitude of builders, engineers, and government agencies. A pair of full-scale 24 x 36 ft slabs were made—one was flat with diagonal reinforcing (Fig. 12) and the other was conventional to serve as a standard. Design dimensions are shown on Fig. 13 and 14.

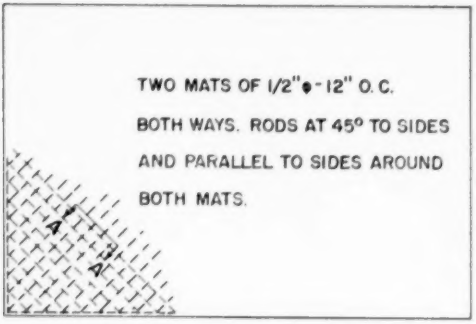
A site was chosen for the two test foundations by sampling the soils on available locations. The particular site selected had an active-type surface and subsoil (Fig. 15). The surface was 3 to 4 ft of black loam (sometimes referred to as "gumbo") while the subsoil was 3 ft of red clay. The slabs were concreted in the early fall while the ground was hot and dry.

A grid of water pipes was buried under diagonally opposite corners of each slab so that moisture could later be introduced in measured quantities.

After curing, steel pins were Ramset into each slab as relative deflection markers, readings being taken with a surveyor's level.

Experimental procedures

Water was injected gradually over a period of several weeks and deflection readings taken. The results have been recorded in Table 6. Though deflection increased steadily as water was injected, schedule requirements made it essential to accelerate the tests. For this reason hydraulic jacks were placed



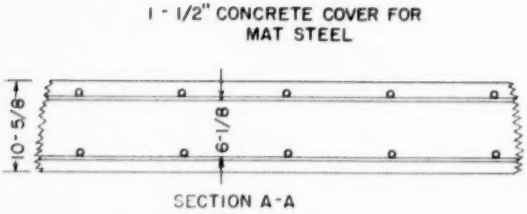


Fig. 14—Diagonally reinforced flat slab

under the *dry* corners and upward loads applied. The corresponding deflections for both slabs are listed in Table 6. A total upward force of 64,000 lb was applied to each slab (32,000 lb per corner). At this point the jack footings failed, and due to lack of funds it was impossible to carry the work further.

Comparison of model and full-scale data

If full-scale and model loading conditions had been identical, the test data would have been directly comparable. Due to the inherent problems of handling the full-scale foundation slab, this was not possible. For this reason logical analytic adjustments have been made to the model results so as to obtain data suitable for comparison.

As explained previously, jacks were applied to the extreme corners of the full-scale slabs while the models were supported on triangular sponge rubber pads. It has been assumed that:

(1) The sponge rubber supports can be replaced by equivalent concentrated loads.

(2) The equivalent reaction for the flat slab acts through the centroid of the *area* of the rubber triangle and is located at one-third the height. This is justified since the flat slab is in contact over the entire surface of the rubber.

Fig. 15—Soil cracks at full-scale slab site

(3) The equivalent reaction for the conventional slab acts through the centroid of the slant *lines* forming the rubber triangle and is located at one-half the height. This is justified since the perimeter beams of the conventional slab have line contact with the rubber.

(4) The slab acts as a simply supported beam bending along its diagonal.

The following effective beam lengths are then applicable:

Conventional slab model test	34.8 in.
Flat slab model test	32.0 in.
Total diagonal length, corner to corner	43.2 in.

From assumptions (2) and (3) above the measured model deflections should be adjusted in proportion to the cube of the effective beam lengths to determine the values that would have been obtained if the model slabs had been supported by the extreme corners. These deflections must then be multiplied by the scale factor 12 to obtain equivalent full-scale information. Finally, to standardize the effect of slab weight, it is desirable to divide the load by the deflection and obtain an effective stiffness for comparison.

TABLE 6—FULL-SCALE DEFLECTION MEASUREMENTS

Condition	Deflection, in.	
	Conventional	Flat (diagonal)
400 gal.*	0.66	0.52
800 gal.*	1.56	1.09
64,000 lb†	3.19	2.00

*Quantity water injected under each of two diagonal corners.
†Total reactionary force applied at two diagonal corners.

Thus, for the conventional slab and following the foregoing procedure and using data from Tables 3 and 4, it is seen that

$$\text{Adjusted model deflection for support at extreme corners} = 0.257 \times \left(\frac{43.2}{34.8}\right)^3 = 0.493 \text{ in.}$$

$$\text{Predicted deflection of full scale under its own weight} = 0.493 \times 12 = 5.90 \text{ in.}$$

$$\text{Predicted stiffness of full scale} = \frac{90{,}000}{5.90} = 15{,}200 \text{ lb per in.}$$

Applying the same procedure to the flat slab model and also the full-scale slabs under 64,000-lb jacking loads, the comparative data of Table 7 were obtained.

It must be frankly admitted that the full-scale tests were extremely crude as far as quantitative information is concerned. Qualitatively, the effect of adding moisture unequally was clearly demonstrated, and the superior performance of the flat slab was made evident. However, no information could be obtained with regard to the actual distribution of load under the two slabs, and therefore, quantitative comparison of their stiffness was not possible under the water flooding test.

The jacking test also provided inadequate data for quantitative comparison. It was known only that the total soil reaction was reduced by the amount of the jacking force, but the subsequent distribution of the remaining soil reaction was completely unknown. It had been hoped that the entire slab could have been lifted, in which case the loading condition would have been accurately determined. It is somewhat remarkable that the model results when adjusted for comparison with full scale show such close agreement as

TABLE 7—COMPARISON OF MODEL AND FULL-SCALE RESULTS

Design	Model stiffness converted to full-scale units, lb per in.	Measured full-scale stiffness, lb per in.
Conventional	15,200	20,000
Flat (diagonal)	31,900	32,000

indicated in Table 7. This proved that the model tests could be used to qualitatively predict the performance of full-scale slabs, but more precise statements cannot be justified.

CONCLUSIONS

Reinforced concrete model slabs can be successfully made and tested with reinforcing rods as small as 1/16-in. diameter and with concrete thickness down to ¼ in. The models can be made for roughly one-fourth of the cost of the corresponding full-scale foundation.

Such models can be laboratory tested under controlled and well organized conditions for a negligible fraction (probably well under 10 percent) of the cost of corresponding tests under full-scale conditions.

Model results are qualitatively in agreement with full-scale data; however experimental uncertainties in the full-scale testing were greater than the discrepancies observed between model and full scale.

Torsional flexibility was found to be the most serious structural defect in monolithic slab foundations. Thickening of the foundation near its center and diagonal placement of reinforcing steel were found to be the two most significant design changes in the correction of this condition. A flat slab with diagonal reinforcing steel and estimated to be of the same cost as the conventional perimeter beam type slab was found to be 2.69 times as stiff.

ACKNOWLEDGMENT

The research and studies forming the basis for this report were performed pursuant to a contract with the Office of the Administrator, Housing and Home Finance Agency, authorized by Title III of the Housing Act of 1948, as amended. The substance of such research and studies is dedicated to the public. It is understood that the accuracy of statements or interpretations contained herein are solely the responsibility of the authors.

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Design and Control of Municipal Paving Concrete*

By ROBERT A. BURMEISTER†

SYNOPSIS

Specifications for concrete together with design and control methods as practiced by the City of Milwaukee for pavement construction in the past 10 years are described. Compression strength tests of 3062 — 6 x 12-in. cylinders are summarized for the period 1949-1953 and air content tests numbering 337 are reported beginning with 1951 when air tests were instituted as a control measure. The cement factor specification is preferred by Milwaukee because of the precise bidding basis afforded.

INTRODUCTION

While technical literature abounds in descriptions of mix proportioning methods and quality control procedure for concrete, translating them into practical municipal field work poses administrative problems involving engineering staff available, laboratory facilities, number of projects, and number of contractors. How the City of Milwaukee has handled these problems in its recent large paving program is described.

Table 1 gives the concrete placed yearly from 1949-1953. Only the 6 sack per cu yd concrete (air-entraining cement) is included in this table. Other concrete such as 4½ sack per cu yd base concrete for bituminous surfaced pavements and concrete for retaining walls, sewers, foundations, and abutments are not included for simplicity of treatment of data.

SPECIFICATION TYPES

Considering the quality of the concrete alone, a municipality constructing concrete pavement can issue specifications of any of three main types:

TABLE 1—QUANTITIES OF SIX-SACK CONCRETE PLACED—1949-1953

Year	Pavement only, cu yd	Sidewalk, curb and gutter, driveway, cu yd	Total, cu yd
1949	49,697	23,077	72,774
1950	38,319	26,783	65,102
1951	61,810	29,645	91,455
1952	64,548	26,910	91,458
1953	75,943	31,678	107,621
		Grand total	428,410

Notes: 1. Number of contractors engaged in this work varied from 16 to 22 yearly.
2. Number of ready-mixed concrete plants engaged in this work varied from 10 to 15.

*Presented at the ACI 51st annual convention, Milwaukee, Wis., Feb. 23, 1955. Title No. 51-49 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 26, No. 10, June 1955, *Proceedings* V. 51. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1955. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Materials Engineer, City of Milwaukee Testing Laboratory, Milwaukee, Wis.

1. Simply call for a portland cement concrete pavement of "adequate" or satisfactory quality.
2. Stipulate various properties of fresh and/or hardened concrete such as W/C, compressive strength, mix by volume or combinations thereof.
3. Stipulate batch quantities for a minimum cement factor and a maximum slump.

The first is plainly inadequate as experience has shown that wide disagreements would occur immediately among contractors themselves and others involved as to what constitutes adequate or satisfactory quality. However, this method is much used for private driveways, walks, and floors. The contractor alone is responsible for proportions of materials and quality of concrete.

The second method, while frequently used, places some responsibility on the contractor and some on the party issuing the contract (depending on how the specifications are worded). But the most important and costly item in the mix—the cement—is not necessarily fixed in amount, hence ready-mixed concrete suppliers or contractors bidding on this specification may differ on what richness of mix should be used.

The third method, and the one used by the City of Milwaukee for the past 12 years, places the responsibility for concrete quality on the engineers representing the city and who design the mix. This method has proved popular with contractors because of the precise bidding basis afforded by the fixed cement content. In writing specifications for concrete using the fixed cement content (a unit cement factor) the principal steps involved are:

1. Selection of cement factor
2. Specifications for cement and fine and coarse aggregate
3. Proportioning mix
4. Control of concrete in field

CEMENT FACTOR

The cement factor for a municipal pavement should be predicated on type and thickness of slab, probable support offered by subgrade, amount and type of reinforcement, degree of workability or consistency required, and service record of aggregates. If no service record is available, mixes must be designed well in advance of construction so that out of mixes of varying richness the optimum cement content can be chosen. Due consideration of all these variables plus a safety factor led Milwaukee to decide on a cement content of 6 sacks per cu yd of freshly mixed concrete.

SPECIFICATIONS FOR CEMENT AND AGGREGATES

Cement

Air-entraining portland cement (ASTM Specification C 175-51T) is used with the following amendment: "The Commissioner* reserves the right to reject an air-entraining cement which does not produce satisfactory air entrainment in concrete. Air entrainment shall be between 3 and 7 percent

*All contracts are written in the name of the Commissioner of Public Works.

by volume when such concrete is made with 6 sacks of air-entraining portland cement per cubic yard of concrete, has a slump of 2 in. to 3 in. and is made in accordance with these specifications."

Fine aggregate

Fine aggregate specification is ASTM C 33-52T with various minor modifications. Grading stipulated is

Passing	Percentage by weight
$\frac{3}{8}$-in. sieve	100
No. 4 sieve	95–100
No. 16 sieve	45–80
No. 50 sieve	7–30
No. 100 sieve	0–6

Coarse aggregate

Coarse aggregate specification is ASTM C 33-52T, also with minor modifications. Two sizes are specified as shown below.

Sieve sizes	Percentage passing by weight
Size No. 1	
1-in. (square openings)	100
$\frac{3}{4}$-in. (square openings)	90–100
$\frac{3}{8}$-in. (square openings)	20–55
No. 4 (square openings)	0–5
No. 8 (square openings)	0–3
Size No. 2	
2-in. (square openings)	100
1$\frac{1}{2}$-in. (square openings)	90–100
1-in. (square openings)	20–55
$\frac{3}{4}$-in. (square openings)	0–15
$\frac{1}{2}$-in. (square openings)	0–5

Forty percent of No. 1 size and 60 percent of No. 2 are the usual proportions of these two items.

MIX PROPORTIONING

Since a cement factor of 6 sacks per cu yd has been chosen for Milwaukee pavement concrete, mix proportioning usually is a matter of determining total aggregate content per cubic yard plus determining the proportion of fine aggregate in total aggregate and water content for the specified 3-in. slump. An experienced concrete technologist can frequently get an accurate mix with just one or two trial batches based on estimated quantities for the desired cement content. Minor adjustments may be needed when production is started. But a more efficient method is to set up three trial batches with cement contents above and below the desired amount. Thus for a minimum cement factor of 6 sacks per cu yd trial batches are set up for a given combination of aggregates and cement for 5, 6, and 7 sacks per cu yd.

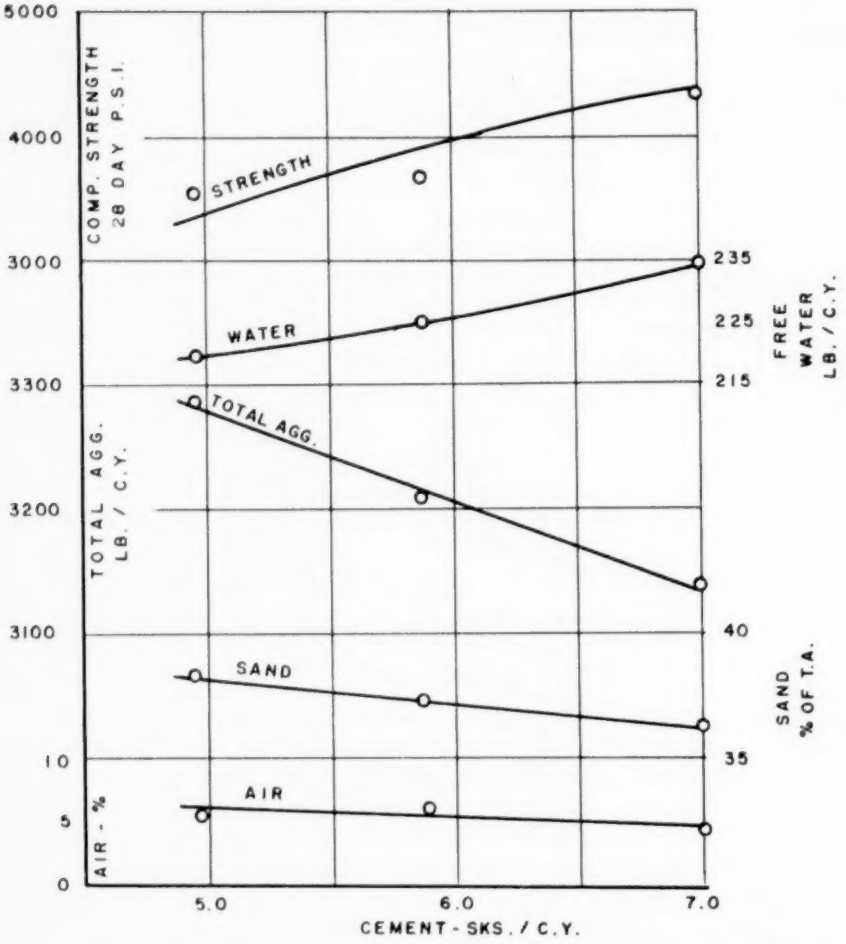

Fig. 1—Concrete mix proportioning chart for ready-mixed concrete—Air-entraining cement Brand No. 1, local sand and gravel. Slump 1¾ to 2¾ in.

These trial batches may be made in the testing laboratory on a small scale (50- to 100-lb batches) or they may be made in the field using regular production equipment. The latter method is preferred because it completely eliminates the need for duplicating field action in producing air entrainment. In addition, check runs can be made for a batch quickly and conveniently.

Procedure for field trial batches

1. Tentative batch quantities for each cement content are computed. Knowing the general characteristics of a given source of aggregate the total amount of oven dry aggregate required for a cubic yard of concrete is estimated. This is then separated into two components—fine aggregate and coarse aggregate—by estimating percent of fine aggregate required. The first estimate—that of total aggregate—is relatively easy (often such information is available in published tables such as those in the report of ACI Committee 613). The second estimate—percent fine aggregate of total aggregate—is not so easy because particle shape, variation of gradation (even though gradation is within specification limits), and potential air entrainment of cement, all have to be taken into account. Limits for percent fine aggregate in the Milwaukee area are 34 to 44 percent when aggregates range from rounded gravel pebbles to crushed stone. Oven dry aggregate batch weights are increased to compensate for surface water and mixing water is adjusted correspondingly.

2. Scales and all equipment are checked for proper functioning. It is particularly important to see that there is no "hold out" of materials in the hoppers or chutes. Likewise the mixer should have some batches run through it so as to coat the interior with mortar.

3. The trial batch is made and if it is too wet, or if percent fine aggregate of total aggregate is not adequate for workability the batch is discarded. The procedure is repeated until a satisfactory batch with correct slump and workability is obtained.

4. Aggregate samples are taken from the hopper chutes as batches are being weighed and are placed in airtight friction-top cans. Aggregate samples are taken to the testing laboratory for tests of moisture content, absorption, specific gravity, and sieve analysis.

5. The fresh concrete from the satisfactory batch is tested on the spot for slump, air content, specific weight and 6 x 12-in. cylinder compression specimens are made.

6. The procedure is repeated for the several selected cement contents. It does not matter if the exact cement contents designed for are not obtained for each batch because when the data are reduced* and plotted as shown in Fig. 1 the desired cement content design can be obtained by interpolation, *i.e.*, batch quantities (on an oven dry basis) can be taken from this chart for the desired cement content. For field production these quantities are increased to allow for moisture in aggregates on the basis of actual tests or estimated moisture. The water added at the mixer may be calculated

Fig. 2—Making slump tests

*Complete calculations are shown in Appendix A.

when moisture tests are made but (as is frequently done) when the moisture in aggregates is estimated the water added at the mixer is that amount which will give the required slump, which for the hand placement methods generally used is specified at 3 in. maximum.

Since the contractor is required to make concrete with batch quantities determined by the city, he is not required to meet a compression strength requirement. However, the specifications give an estimated strength of 3600 psi.

FIELD CONTROL—COMPRESSION TESTS

Beside the customary inspection staff at the ready-mixed concrete plant and project site (currently all Milwaukee paving concrete is ready-mixed) the city testing laboratory exercises another control and that is to make field 6 x 12-in. compression specimens, air content tests, and slump tests. Field testing on a typical paving project is shown in Fig. 2 to 5.

Test cylinder molds are of paraffined paper. The cylinders are field cast in groups of four, one for a 7-day test and three for the 28-day test. Cylinders are allowed to stand in the field for 24 hr with no protection and are then taken to the laboratory moist closet. Because paper molds give results lower than with metal molds* and since the treatment of the field cast cylinders are not in full compliance with ASTM standards, the strength results are lower than if ASTM requirements were fully complied with. However, the paper molds are economical and make possible a larger number of tests than if metal molds were used. It has been the writer's experience that a large number of tests made at frequent intervals (even though of a lower order of accuracy than possible) give better control results than highly accurate but fewer tests widely spaced. The effect of frequent tests on workers'

Fig. 3—Making test cylinders

*Burmeister, Robert A., "Tests of Paper Molds for Concrete Cylinders," ACI JOURNAL, Sept. 1950, *Proc.* V. 47, pp. 17-24.

Fig. 4—Filling air meter with concrete

morale is alone of great benefit. Table 2 gives compression strengths of cylinders taken during the past 5 years of paving. These strengths have been tabulated by ready-mixed concrete plants. Note that the average for the 5 years is 3680 psi (28-day compressive strengths). Since all test specimens are made at unannounced times, this average shows that there was excellent compliance with specifications.

For most projects slump tests are made only occasionally as a check on estimated water content, but if there is disagreement on the job, slump and other tests such as specific weight of fresh concrete are made.

FIELD CONTROL—AIR CONTENT TESTS

Air contents are determined in the field by use of a Press-UR-Meter. No corrections are made for absorption of aggregates because of the small amount and questionable validity of such corrections. Table 3 gives results of the air-content test program instituted in 1951. Note that the average air con-

Fig. 5—Reading air content of concrete

tents for each year has been highly satisfactory and there were relatively few tests outside the 3-7 percent specified. In all cases air was entrained by use of air-entraining cements and no air entrainment depressors or accelerators were used.

TABLE 2—COMPRESSIVE STRENGTHS OF PAVING CONCRETE*

Code No. of ready-mixed concrete plant	1949		1950		1951		1952		1953	
	Avg strength	No. of cylinders	Avg strength	No. of cylinders	Avg strength	No. of cylinders	Avg strength	No. of cylinders	Avg strength	No. of cylinders
1	3810	39	—	—	4150	9	4400	15	3860	6
2	3470	57	3860	60	3640	71	3650	51	3780	72
3	—	—	—	—	—	—	—	—	3490	9
4	3010	48	3330	122	3550	138	3390	108	3500	93
5	—	—	—	—	—	—	—	—	3610	63
6	3680	6	3840	126	3700	62	3720	90	3780	120
7	3580	12	3590	42	3400	78	3510	69	3430	117
8	4280	15	4030	81	3830	87	3950	69	3930	42
9	—	—	—	—	—	—	—	—	4540	21
10	3990	21	4270	33	4210	90	4400	81	3640	75
11	—	—	3740	30	3240	27	3760	39	3440	21
12	—	—	3480	6	4050	51	4260	9	3920	12
13	2790	6	—	—	3360	12	3540	30	3500	93
14	3610	33	3580	45	3660	18	3440	24	3320	33
15	3830	15	3590	90	3350	78	3690	58	3640	34
	3560	252	3710	635	3680	721	3750	643	3630	811

*28-day strengths of moist cured 6 x 12-in. cylinders. 6 sack per cu yd air-entrained concrete. Average for 5 years and 3062 cylinders = 3680 psi.

Appendix A—Computations

Batch weights wet (Nominal 1 cu yd), lb

	Batch 1	Batch 2	Batch 3
Cement	564	470	658
Fine aggregate	1225	1300	1175
Coarse aggregate No. 1	825	825	800
Coarse aggregate No. 2	1225	1235	1225
Water (added)	195	180	204
Total	4034	4010	4062

Total moisture in aggregates (oven dry)

	Batch 1	Batch 2	Batch 3
Fine aggregate	2.1 percent	3.2 percent	2.6 percent
Coarse aggregate No. 1	1.7 percent	2.0 percent	1.8 percent
Coarse aggregate No. 2	2.0 percent	1.4 percent	1.5 percent
Fine aggregate	$\frac{1225\times0.021}{1.021} = 25$	$\frac{1300\times0.032}{1.032} = 40$	$\frac{1175\times0.026}{1.026} = 30$
Coarse aggregate No. 1	$\frac{825\times0.017}{1.017} = 14$	$\frac{825\times0.020}{1.020} = 16$	$\frac{800\times0.018}{1.018} = 14$
Coarse aggregate No. 2	$\frac{1225\times0.020}{1.020} = 24$	$\frac{1235\times0.014}{1.014} = 17$	$\frac{1225\times0.015}{1.015} = 18$
Total, lb	63	73	62

TABLE 3—FIELD TESTS FOR AIR CONTENT

Cement	1951					1952					1953				
	No. of tests	No. of tests			Avg of all tests, percent	No. of tests	No. of tests			Avg of all tests, percent	No. of tests	No. of tests			Avg of all tests, percent
		Under 3 percent	Over 7 percent	Over 8 percent			Under 3 percent	Over 7 percent	Over 8 percent			Under 3 percent	Over 7 percent	Over 8 percent	
Brand 1	44	1	2	1	5.3	23	0	0	0	5.0	30	0	0	0	4.8
2	12	0	6	0	7.0	8	0	3	0	6.4	18	1	4	0	6.0
3	6	0	0	0	5.9	19	0	1	0	5.5	6	0	0	0	5.4
4	15	1	0	0	4.6	29	0	6	5	6.2	20	0	0	0	4.8
5	38	1	3	0	5.7	32	0	0	0	5.5	37	0	5	2	6.0
Total	115					111					111				
Avg percent					5.7					5.7					5.4

Batch weights, dry

	Batch 1	Batch 2	Batch 3
Fine aggregate	1225 − 25 = 1200	1300 − 40 = 1260	1175 − 30 = 1145
Coarse aggregate No. 1	825 − 14 = 811	825 − 16 = 809	800 − 14 = 786
Coarse aggregate No. 2	1225 − 24 = 1201	1235 − 17 = 1218	1225 − 18 = 1207
Total, lb	3212	3287	3138

Total water in and on aggregates

	Batch 1	Batch 2	Batch 3
Fine aggregate	25	40	30
Coarse aggregate No. 1	14	16	14
Coarse aggregate No. 2	24	17	18
Total, lb	63	73	62

Water absorbed by aggregates

Fine aggregate—Obtained by air drying as-received sample to particle saturated, surface dry condition; weighing, then oven drying, and again weighing.

Coarse aggregate—Obtained by soaking in water 24 hr, then drying with towel; weighing, oven drying, and again weighing.

Fine aggregate—0.9 percent on composite sample.

Coarse aggregate—1.1 percent on composite sample of 40 percent No. 1 and 60 percent No. 2.

	Batch 1	Batch 2	Batch 3
Fine aggregate	1200 × 0.009 = 11	1260 × 0.009 = 11	1145 × 0.009 = 10
Coarse aggregate	2012 × 0.011 = 22	2027 × 0.011 = 22	1993 × 0.011 = 22
Total, lb	33	33	32

Free water from aggregates

Batch 1	Batch 2	Batch 3
63 − 33 = 30 lb	73 − 33 = 40 lb	62 − 32 = 30 lb

Water-cement ratio computations

	Batch 1	Batch 2	Batch 3
Free water from aggregates	30	40	30
Water added at mixer	195	180	204
Total free water for W/C, lb	225	220	234
W/C, by weight	$\frac{225}{564} = 0.399$	$\frac{220}{470} = 0.468$	$\frac{234}{658} = 0.356$
W/C, by volume	0.399×1.5 = 0.598	0.468×1.5 = 0.702	0.356×1.5 = 0.534
W/C, gal. per sack	0.598×7.5 = 4.48	0.702×7.5 = 5.26	0.534×7.5 = 4.00

Absolute volumes, cu ft

	Batch 1	Batch 2	Batch 3
Cement	$\frac{564}{3.15\times62.4} = 2.87$	$\frac{470}{3.15\times62.4} = 2.39$	$\frac{658}{3.15\times62.4} = 3.35$
Fine aggregate	$\frac{1200}{2.61\times62.4} = 7.37$	$\frac{1260}{2.61\times62.4} = 7.74$	$\frac{1145}{2.61\times62.4} = 7.03$
Coarse aggregate	$\frac{2012}{2.71\times62.4} = 11.90$	$\frac{2027}{2.71\times62.4} = 11.99$	$\frac{1993}{2.71\times62.4} = 11.79$
Water	$\frac{225}{62.4} = 3.61$	$\frac{220}{62.4} = 3.53$	$\frac{234}{62.4} = 3.75$
Absolute volume of batch	25.75	25.65	25.92
Fresh weight per cu ft of concrete, field test, lb	146.48	147.36	150.40
Fresh weight per cu yd, lb	$146.48\times27 = 3955$	$147.36\times27 = 3979$	$150.40\times27 = 4061$
Batch weight, lb	4034	4010	4062
∴ Yield, cu yd	$\frac{4034}{3955} = 1.0200$	$\frac{4010}{3979} = 1.0078$	$\frac{4062}{4061} = 1.0002$
Yield, cu ft	$27\times1.0200 = 27.540$	$27\times1.0078 = 27.211$	$27\times1.0002 = 27.005$
Cement content, sacks per cu yd	$6/1.0200 = 5.88$	$5/1.0078 = 4.96$	$7/1.0002 = 7.00$

Air content, gravimetric

	Batch 1	Batch 2	Batch 3
Actual volume	27.540	27.211	27.005
Absolute volume	25.75	25.65	25.92
Air volume, cu ft	1.790	1.561	1.085
Air voids, percent	$1.790/27.540 = 6.5$	$1.561/27.211 = 5.7$	$1.085/27.005 = 4.0$
Air voids, field test, pressure method, percent	6.0	5.8	4.3

Title No. 51-50

Tobermorite and Related Phases in the System CaO-SiO_2-H_2O*

By GEORGE L. KALOUSEK†

SYNOPSIS

The hydrothermal reactions at temperatures of 125 to 175 C between $Ca(OH)_2$ and quartz or silicic acid have been studied in detail with regard to the reaction products to form particularly in the range of molar compositions of 0.8 CaO: SiO_2 to 1.25 CaO: SiO_2. The solid phases were studied by x-ray, DTA, BET nitrogen adsorption, electron microscopy and chemical analysis. The products prepared from mixtures made with silicic acid were fibrous in form and ranged in composition from 0.8 to 1.25 C/S. The same or closely related solids were also prepared with quartz. The fibrous $C_6S_5H_n$ transformed to tobermorite $C_4S_5H_5$, a platy crystalline solid. The transformation was very rapid in mixes made with quartz but comparatively slow in mixtures made with silicic acid. Although the fibrous and platy phases are similar, the differences between the two, particularly in physical properties, appear to be real. The fibrous phase does not integrate and, upon drying, undergo a marked shrinkage. The platy phase integrated into dimensionally stable solids of superior strengths.

INTRODUCTION

The results of the investigations conducted in the General Research Division of the Owens-Illinois Glass Co. on the synthesis of the mineral tobermorite ($C_4S_5H_5$)‡ and related solids in the system C-S-H appear to be of general interest in the chemistry and morphology of the hydrous calcium silicates. Data from these studies on some of the intermediate phases have been published.[1] Also, methods of synthesizing tobermorite are described in a recent patent,[2] the application for which was filed in 1948. Subsequent to the date of filing, other pertinent data bearing on the low-lime phases in the system have been obtained. It appeared that these later data, together with further details on the course of the reactions described in the patent and a review of the published data, would be worthwhile additions to the general chemistry of hydrous calcium silicates and therefore are presented in this report.

METHODS OF SYNTHESIS

The solid phases were prepared at temperatures of 25 C and 125 to 175 C in closed containers and in the presence of liquid water. At ordinary temper-

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‡The usual abbreviations are used for expressing the formulas of the oxides, *viz.* $CaO = C$; $SiO_2 = S$; $H_2O = H$.

atures the solids were prepared by precipitation from aqueous solutions of silicic acid and calcium hydroxide following the method of Flint and Wells.[3] Compositions of the precipitates were affected by varying the volumes of the solutions of $Ca(OH)_2$ relative to that of silicic acid. The details of tests and results have been published.[1]

Calcium oxide and either silicic acid or finely pulverized quartz (minus No. 325 screen) served as raw materials in the hydrothermal studies at temperatures of 125 to 175 C. The CaO was prepared by igniting low-alkali, reagent quality $CaCO_3$ at a temperature of 1050 to 1100 C for 2 to 4 hr. The dry raw solids in desired proportions were first mixed thoroughly and then with water in a water-to-solids ratio of 3.5 by weight. In an alternate method the CaO was hydrated first in six parts of water (at 50 to 60 C) by weight, the silica-bearing component was then added to this slurry and the composite thoroughly mixed immediately before being placed in small autoclaves for processing. Although nearly all the samples described in this report were made by the first method, the alternate method was used in preparing larger amounts of the solids. The reactions appeared to proceed more rapidly for the mixes made with freshly slaked lime slurry. Otherwise no striking difference was observed between the results for the two methods such as that experienced in the preparation of the crystalline lime rich phases.[4]

Autoclaving was carried out at selected temperatures and for various periods. At selected periods of processing, one autoclave from a group of four to six containing portions of the same mix, was quenched in cold water. The contents were filtered and washed, in order, with acetone and diethyl ether, and finally dried at 110 to 120 C for 1 to 2 hr. The samples were stored in tightly stoppered bottles.

IDENTIFICATION AND CHARACTERIZATION OF SOLID PHASES

The method of differential thermal analysis (DTA) was adopted for characterizing the reaction products, and its application as used in these studies is described elsewhere.[5]

A Phillips' diffraction unit with a 57.3-mm radius powder camera using $Cu\ K$ alpha radiation and nickel filter was employed for recording the x-ray diffraction on photographic film. A Norelco Geiger-tube x-ray spectrogoniometer was also used in these studies.

A model EMU, RCA electron microscope was applied in the studies of the crystal habit of the finely divided materials.

The BET nitrogen adsorption method was used in determination of the surface areas of the solid phases. These results, in conjunction with others, also served the purpose of differentiating between closely related phases in one part of the study.

The extent to which silica and $Ca(OH)_2$ reacted during the earlier stages of the reaction was of considerable interest in this investigation. Accordingly, free lime and unreacted silica were determined quantitatively. The

methods and results of free lime tests are reported elsewhere.[1] In the analysis for unreacted silica, following the method of Steopoe,[6] a finely pulverized sample is rapidly dissolved in 1 N HCl solution containing 5 percent $NaCl$, then washed with a solution of 5 percent Na_2CO_3—5 percent $NaCl$ by weight, and finally washed in succession with 1 percent HCl solution and hot water. The ignited residue represents the amount of unreacted silica. Analysis of mixtures of a sample of fibrous CSH_n and silicic acid, the latter in amounts of 1 to 5 percent by weight of mix, for free silica indicated that the results obtained by the described method were accurate to ± 0.5 percent. Limitations of the method in determinations on well crystallized solids are reported elsewhere.[4]

RESULTS

Early in this investigation, examinations with the electron microscope revealed presence of fibrous and platy crystals in the reaction products. Eventually, procedures were developed by which each of these forms, as described below, was isolated as a homogeneous solid.

Isolation and identification of the fibrous phase

The low-lime solids forming at 25 C in presence of large amounts of solution were found to have compositions ranging from 0.8 C/S to 1.33 C/S. All compositions of the fibrous solids in this range showed nearly the same x-ray results. Patterns of four compositions are included in Table 1, last four columns. These patterns are in good agreement with those published by Taylor[7] for similar preparations which he designated "CSH-I". One of the

TABLE 1—X-RAY DATA OF SYNTHETIC TOBERMORITE AND FIBROUS LIME-SILICA HYDRATES

$4CaO{\cdot}5SiO_2{\cdot}5H_2O$ (tobermorite)		$4CaO{\cdot}5SiO_2{\cdot}nH_2O$ (fibrous)		$CaO{\cdot}SiO_2{\cdot}nH_2O$ (fibrous)		$5CaO{\cdot}4SiO_2{\cdot}nH_2O$ (fibrous)		$4CaO{\cdot}3SiO_2{\cdot}nH_2O$ (fibrous)	
I/I_o*	d,A	I/I_o	d,A	I/I_o	d,A	I/I_o	d,A	I/I_o	d,A
VS	11 2	—	—	VWB	11	VWB	11	VVWB	11
		M	7 3†	W to M	7 2	VW	7 4	—	—
M	5 4	VW	5 4	W	5 4	—	—	—	—
W	3 49								
W	3 30								
VS	3 07	VS	3 04	S	3 05	S	3 02	S	3 03
S	2 96								
S	2 80	W	2 77	W	2 78	—	—	W	2 76
W	2 51								
W	2 41								
W	2 24								
W	2 13								
W	2 06								
W	1 99								
S	1 838	M	1 824	M	1 823	M	1 827	M	1 820
MS	1 671	W	1 659	VW	1 659	VW	1 669	W	1 672
W	1 620								
M	1 531			VVW	1 522				
VW	1 431								
VW	1 389	VVW	1 392	VVW	1 392				

*Estimated relative intensities: S, strong; M, medium; W, weak; V, very; B, broad; I_o being the maximum intensity.

†The 7.3 A line could be considered characteristic of the fibrous products. However, a large number of the samples failed to show this line, and when present, its intensity varied from VW (broad) to S.

important characteristics of the members of the 0.8 to 1.33 C/S series is their stability in the acetoacetic ester medium used for free lime determination.[1] The phases of lime contents higher than that of the 1.33 C/S hydrate are not similarly stable, the lime in part is extractable.

Although x-ray results do not differentiate between the different compositions, the DTA curves indicated reproducible and characteristic differences.[1] The 0.8 to 1.0 C/S hydrates underwent a strong exothermic reaction at 830 ± 5 C, the extent of the change being about twice as large for the 1.0 C/S hydrate compared to that of the 0.8 C/S solid. The 1.25 C/S phase manifested this reaction at 865 ± 5 C, and the 1.33 C/S hydrate at 900 ± 5 C.

The fibrous nature of these phases is illustrated by the electron photomicrograph in Fig. 1. These fibrous crystals were exceedingly fine.

Fibrous products identical to those just described, and extending in composition from 0.8 to 1.25 C/S, were also prepared hydrothermally from mixtures of solid lime and silicic acid at temperatures of 125 to 175 C. In these syntheses the periods of autoclaving, depending on the different factors to be discussed, varied from several weeks at 125 C to a few hours at 175 C. The products obtained from mixtures having C/S ratios higher than 1.25 contained the 1.25 C/S hydrate and the alpha "dicalcium" silicate hydrate.[8] The solids made at 25 C adsorbed about 30 to 40 percent more nitrogen than did those made at 100 C to 175 C. Therefore, it was assumed that the size of the fiber was slightly greater in the samples processed at the more elevated temperatures. Also, the samples prepared at room temperature underwent the exothermic reaction during differential thermal analysis at a temperature approximately 7 C (average of about 60 results) lower than

Fig. 1—Electron photomicrograph of fibrous CSH_n

TABLE 2—CONDITIONS UNDER WHICH THE FIBROUS $4CaO \cdot 5SiO_2 \cdot nH_2O$, $CaO \cdot SiO_2 \cdot nH_2O$, $5CaO \cdot 4SiO_2 \cdot nH_2O$, AND $4CaO \cdot 3SiO_2 \cdot nH_2O$ FORMED FROM MIXES OF LIME AND SILICIC ACID, AND SOME PROPERTIES OF THESE SOLIDS

Composition raw mix, C/S ratio	Autoclave temperature, deg C	Autoclave time	C/S ratio, corrected	Assigned molar comp., $CaO:SiO_2$		Thermal analysis exothermic peak		Surface area, sq m per g
						Temp., deg C	Extent (div.)	
0.8	175	12-16 hr.	0.83	4	5	835	5-20	175
0.8	150	18 hr.	0.84	4	5	830	31	155
1.0	175	15 hr.	1.00	1	1	835	55	—
1.0	150	4 days	1.04	1	1	839	63	—
0.95	150	3 days	0.97	1	1	834	51	—
0.95	150	2 days	1.02	1	1	833	56	125
1.00	125	28 days	—	1	1	833	55	—
1.20	175	15 hr	1.28	5	4	870	35	—
1.20	150	7 days	—	5	4	870	40	90
1.20	140	7 days	1.27	5	4	870	41	—
*4.0 (Soln.)	(25-30)	7 days	1.26	5	4	870	55	160
*6.0 (Soln.)	(25-30)	28 days	1.35	4	3	900	52	94

*Made from aqueous solutions at room temperature.

did those of comparable compositions processed at 175 C. Decrease in the temperature of the exothermic reaction is to be expected with increased fineness of the sample under test.

The water contents (loss on ignition) of the samples dried at 110 to 120 C were approximately 14-15, 15-16, 17-19, and 17-20 percent for the 0.8, 1.0, 1.25, and 1.33 C/S hydrates, respectively. The exact amount of hydrate water of these solids would be difficult to determine because of the exceedingly small size of the crystals, making doubtful the amount of surface adsorbed water; and also more recent reports on the subject indicate the possibility that movement of structural, presumably zeolitic, water may even occur at room temperature.

TABLE 3—RESULTS ON 0.8 C/S SAMPLES MADE WITH SILICIC ACID AND INDURATED AT 175 C INDICATING THE FORMATION OF THE FIBROUS $4CaO \cdot 5SiO_2 \cdot nH_2O$

Time of autoclaving, hr	Free SiO_2 (anhy. basis), percent	Ignition loss, percent	C/S* ratio, corrected	Surface area, sq m per g	Thermal analysis exothermic peak	
					Temp., deg C	Extent (div.)
0.6				68	870	12
3	14.8	14.8	1.06	108	841	58
3.5				122	839	39
5	13.2	14.7	1.02	122	840	51
5				127	839	58
6.5	13.3	15.1	1.02	127	840	48
7	13.0	—	1.01	122	845	57
8	11.3	14.3	0.98	165	840	30
9	7.6	14.8	0.90	156	831	28
10.5	6.5	13.9	0.88	174	836	27
12	3.9	14.0	0.84	180	838	24
13.5	4.0	13.5	0.84	177	835	16
16	—	—	—	177	835	21
17	2.9	15.3	0.82	165	837	16
19	3.2	14.6	0.83	168	839	6
21.5	4.8	14.6	0.85	134	840	5
24	2.8	14.0	0.82	119	842	6
24				69	838	12
48				60	845	2

*Analytical results for free lime and lime as $CaCO_3$ showed that about 0.8 to 1.2 percent of CaO might not be combined with silica. The present results were all corrected for 1.0 percent of CaO, an average value.

The most favorable conditions of temperature and time of formation of the fibrous solids following the procedure described in this report are given in Table 2. The reactions at 125 C are very slow, requiring up to three weeks for completion, and, therefore, most of the tests were conducted at 150 and 175 C. At 175 C the reactions of formation of the fibrous solids are markedly accelerated, being completed in the order of 6 to 14 hr.

At 175 C the fibrous 0.8 C/S hydrate is stable only during a relatively short time. In establishing the conditions for the formation of this phase, series of mixes of 0.8 C/S ratio were autoclaved for different periods of time starting with a period in the order of 1 hr. The samples were analyzed chemically for free silica and studied for change in surface areas and DTA curves. In Table 3 are given the analytical, surface area, and DTA data obtained on samples processed at 175 C. The original composition corrected for the amount of unreacted silica gives the composition of the reaction product which is designated in the table as "C/S ratio, corrected". After about 3 hr, the CSH_n formed and persisted for about 4 hr. Starting at about 7 hr of total autoclaving, the CSH_n underwent a reaction with the residual silica and this was completed at about 12 hr. The product was essentially the fibrous 0.8 C/S hydrate. The variations in the surface area of the prod-

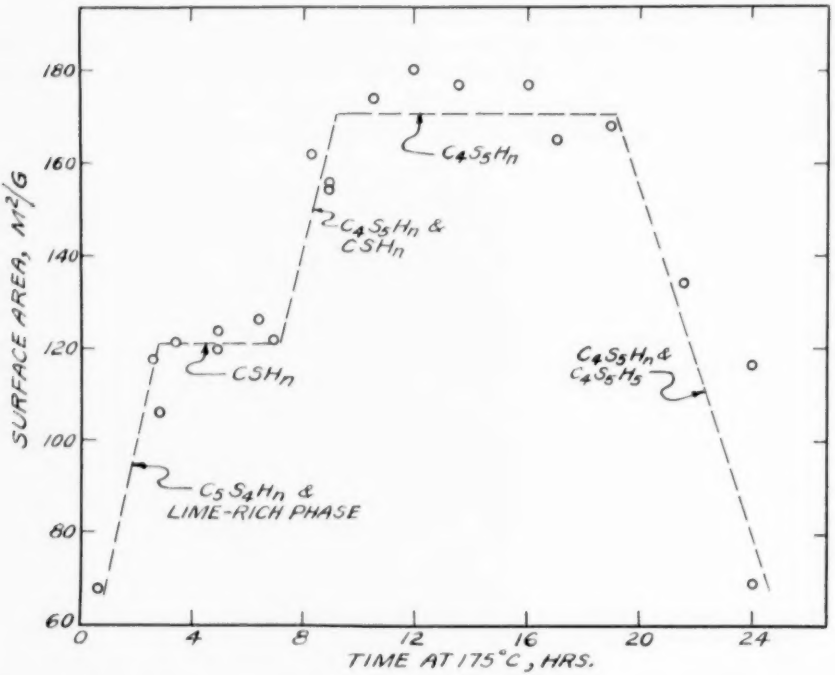


Fig. 2—Relation of surface areas to time of autoclaving of 0.8 C/S mixture made with silicic acid at 175 C

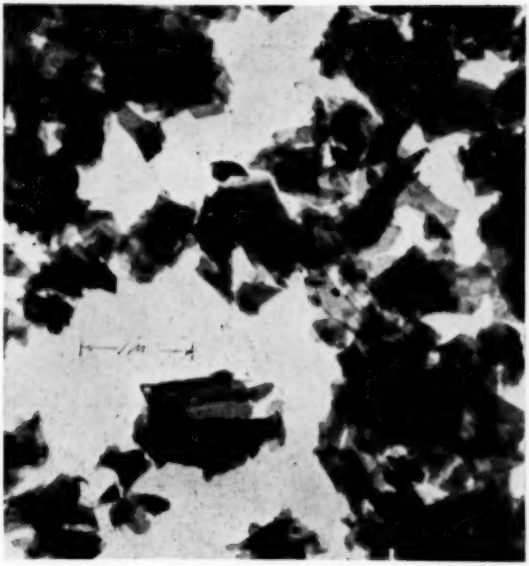


Fig. 3—Electron photomicrograph of synthetic tobermorite

ucts formed at different times are shown in Fig. 2. The 0.8 C/S hydrate had a considerably higher surface area than did the 1.0 C/S product. During the autoclaving interval between 12 to 19 hr the composition, surface area, and DTA results remained essentially constant.

Isolation and identification of tobermorite (platy phase)

The fibrous 0.8 C/S hydrate was not stable for long at a temperature of 175 C during autoclaving. After about 19 hr the reaction product manifested a decrease in surface area and a marked reduction in the extent of the exothermic peak. The x-ray diffraction also underwent significant changes which became more and more pronounced with samples autoclaved for the longer periods. Among these changes were the appearances of the line of strong intensity at 11.1 A, the resolution of the 3.03 ± 0.02 A line into a doublet at 3.07 and 2.96 A, and a small increase in the spacing of the 1.82 A line to about 1.84 A. The x-ray pattern for the fully crystallized sample is given in Table 1. This sample consisted only of the platy phase. An electron microscope view is shown in Fig. 3.

Tobermorite, which is called Lepisil in the patent,[2] is much more readily prepared from quartz, or a pure form of tripoli, than with silicic acid. Solids higher in lime, up to 1.0 C/S ratio, have been prepared which gave identically the same x-ray pattern as tobermorite. Presumably, these may also form even with higher lime content; however, the formation of the alpha "dicalcium" silicate hydrate generally results. The formation of members of the tobermorite series of the higher lime contents requires lower temperatures and considerably longer time of processing. At the higher temperatures used in practice, the formation of xonotlite with tobermorite occurs.

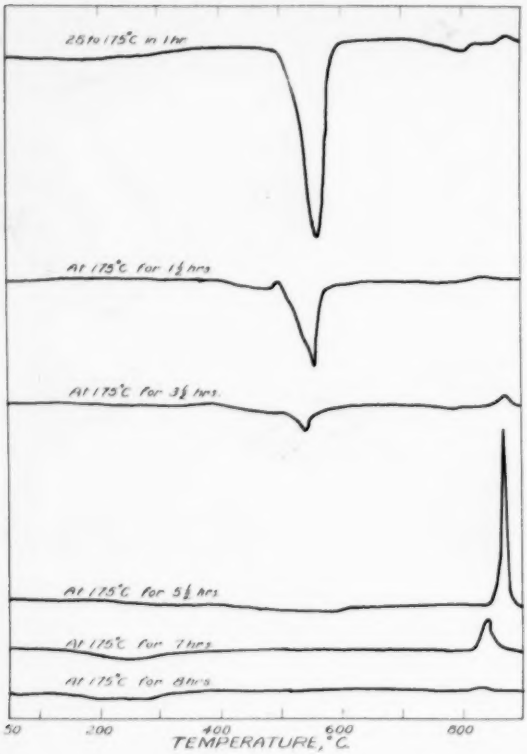


Fig. 4—DTA results on products prepared at different times of autoclaving at 175 C from 0.8 C/S mixture made with quartz

Reactions between $Ca(OH)_2$ and quartz, and $Ca(OH)_2$ and silicic acid

The development of the subject matter of reactions between $Ca(OH)_2$ and different forms of silica at 125 to 175 C requires a brief description of the lime-rich phases, which are the first products formed in the reaction. It has been indicated[2] that three different kinds of lime-rich products may appear as the first intermediate phases. Recently, these three phases have been identified more definitely. The most common form, and readily identified by x-ray, is the lime-rich analogue of the fibrous 0.8 to 1.25 C/S hydrates. It is generally formed from silicic acid and lime. The second type has been described[8] as a poorly crystallized form of the alpha "dicalcium" silicate hydrate. It is formed most frequently as the first product in the hydrothermal reaction of lime and quartz. A third type, apparently a poorly crystallized form of the mineral hillebrandite, appears to form in portland cement pastes at temperatures of about 80 to 125 C.

Mixes of 0.8 C/S ratio made of lime and either quartz or silicic acid were processed at 175 C for different periods, and the products of the reactions at the various stages were studied by DTA, x-ray, nitrogen adsorption, and analysis for unreacted silica. The DTA results for mixes made with quartz are given in Fig. 4. The curve at the top of the figure is essentially that of

$Ca(OH)_2$; only a small amount of the lime-rich phase formed during the heating-up period. During the following period of 1.5 hr at 175 C about two-thirds of the total $Ca(OH)_2$ had reacted to form the poorly crystallized alpha-type hydrate. The next DTA curve, after 3.5 hr, shows a near depletion of the $Ca(OH)_2$. The reaction product was the 1.75 C/S alpha-type hydrate in poorly crystallized form.

The reaction does not stop when all the free $Ca(OH)_2$ had reacted. In the present mixes of 0.8 C/S ratio, less than one-half of the total quartz, as shown analytically by determination of free silica, was combined at the time when the last trace of free $Ca(OH)_2$ reacted. The subsequent reaction, therefore, occurs between the lime-rich phase and the residual quartz. The lime-rich solid on basis of chemical analysis showed a composition of about 1.8 C/S.[8] Samples of the product prepared at ordinary temperatures manifested a C/S of 1.7 to 1.8. Although, due to experimental difficulties, the composition of this phase could not be established unequivocally, the composition is probably close to a ratio of 1.75 C/S and the formula, $C_7S_4H_n$, corresponding to this ratio is used in the reactions given in the following discussion.

The DTA curves in Fig. 4 (those in Fig. 5 to be discussed) and others not reproduced in this report showed a definite and reproducible sequence in the formation of the solids of different compositions described in the preceding sections. The first step of the over-all reaction

$$7C + 4S + \text{aq.} \rightarrow C_7S_4H_n \qquad (1)$$

represents the combination of all the free $Ca(OH)_2$ in the formation of the lime-rich phase. The other stages, in the following order, represent the reactions between the different hydrous calcium silicates and the residual silica

$$5C_7S_4H_n + 8S \rightarrow 7C_5S_4H_n \qquad (2)$$

$$C_5S_4H_n + S \rightarrow 5CSH_n \qquad (3)$$

$$4CSH_n + S \rightarrow C_4S_5H_n \qquad (4)$$

The completion of the particular reaction under consideration is represented by the conversion of the fibrous $C_4S_5H_n$ to tobermorite

$$C_4S_5H_n \rightarrow C_4S_5H_5 \qquad (5)$$

The curve at the bottom of Fig. 4 is that of a well-crystallized sample of tobermorite. This curve shows only a small endothermic effect at about 260 C. The exothermic reaction does not occur in DTA studies of this solid.

The studies of a parallel series made with silicic acid show interesting differences in the nature of the lime-rich phase and the time required for the formation of the various solids compared to those made with quartz. The DTA curves are presented in Fig. 5, and the analytical data, surface area results, and summary of DTA data are given in Table 3. The use of silicic acid, instead of quartz, made it possible to isolate the fibrous $C_4S_5H_n$ at elevated temperature (150 and 175 C).

As shown by the top curve in Fig. 5, the $Ca(OH)_2$ combined completely with the silicic acid in the heating-up period of 40 min. X-ray examination of this solid showed weakly the principal lines of the 0.8 to 1.25 C/S hydrates

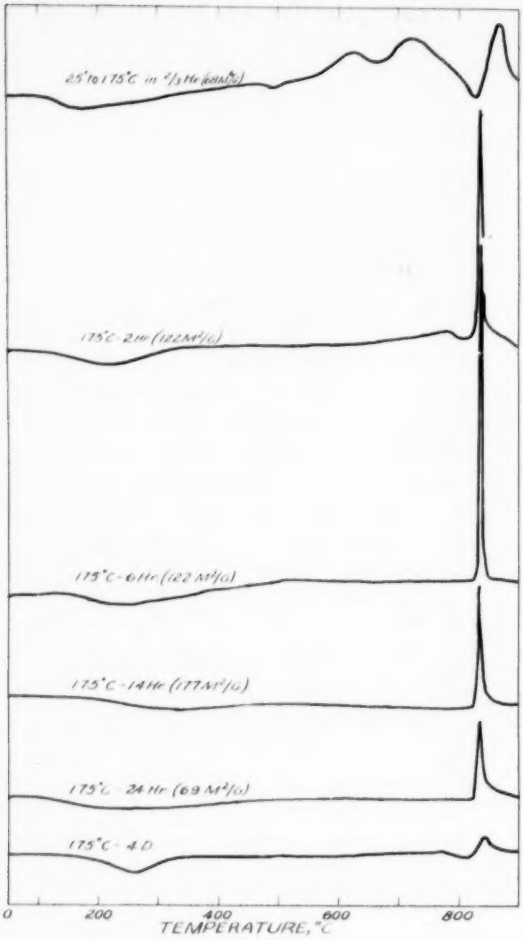


Fig. 5—DTA results on products prepared at different times of autoclaving at 175 C from 0.8 C/S mixture made with silicic acid

and broad bands, and it apparently is the lime-rich analogue of these hydrates. The combination of the lime-rich phase with the residual silica was very rapid. The intermediate $C_5S_4H_n$ could not be isolated entirely free of either the $C_7S_4H_n$ or its transformation product, the CSH_n. The CSH_n formed in about 2 hr and persisted as such up to about 7 hr of autoclaving. It is interesting to note that the surface area of this phase remained nearly constant at about 120 sq m per g. After about 7 hr the reaction with the residual silica became rapid and, in about 9 to 10 hr from the start of test, the $C_4S_5H_n$ was present essentially as a pure phase with a surface area of about 170 to 180 sq m per g. After a total time of about 18 to 19 hr, the transformation to tobermorite began. At first it was rapid but complete conversion generally was not obtained for about four to seven days.

The essential difference between the reactions of $Ca(OH)_2$ and quartz, and $Ca(OH)_2$ and silicic acid is one of rates of reaction. The rates of reaction are shown diagrammatically by a plot of the composition of the reaction products (C/S ratios) and the time of autoclaving in Fig. 6. This figure is a summary of the complete data used in the preparation of Fig. 4 and 5. The most important feature, as mentioned, is the relative persistence of the CSH_n and the $C_4S_5H_n$ in the mixes made with silicic acid compared to those made with quartz. Before commenting on the difference, it should be mentioned that the times given in Fig. 6 are only approximate. Changing the mixing technique, for example, or the brand of silicic acid resulted in differences of elapsed autoclaving time to the various phases of as much as 3 hr. Also, the reactions were not always completely consecutive; simultaneous formation of two solids occurred such as the appearance of $C_5S_4H_n$ and CSH_n as products of the decomposition of the lime-rich gel in samples made of silicic acid. In these mixtures exothermic peaks at 835 ± 5 C and 870 ± 5 C served to indicate presence of these highly similar solids.

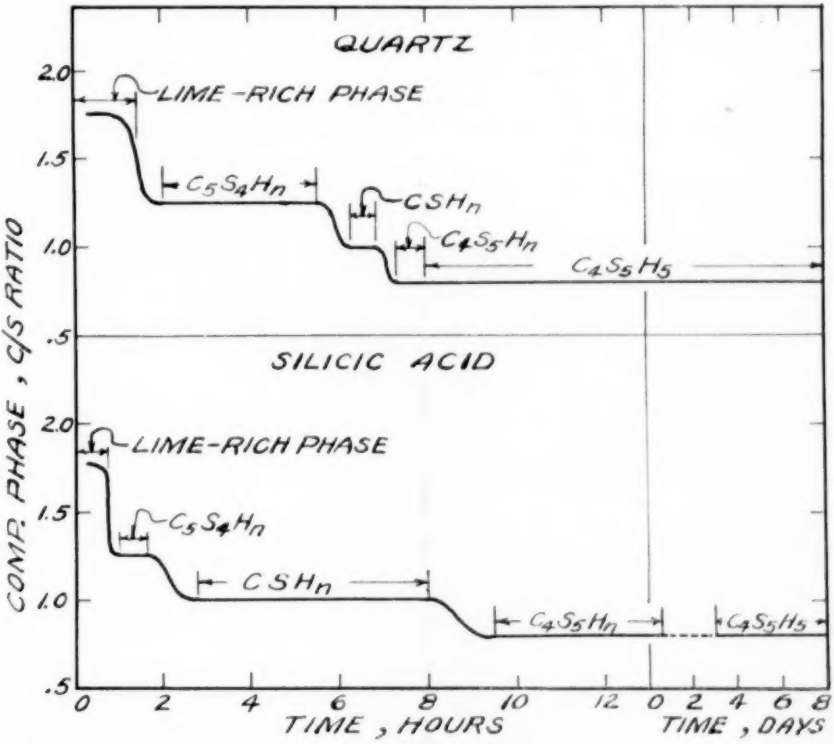


Fig. 6—Time required to form the different products from 0.8 C/S mixes of $Ca(OH)_2$ and quartz or silicic acid autoclaved at 175 C

Comparison of results on fibrous and platy phases

Silicic acid is, of course, much more reactive with lime than is quartz, and the initial reactions are therefore more rapid. In view of this greater reactivity, it is surprising that the formation of tobermorite (actually the transformation of the fibrous $C_4S_5H_n$ to this phase) was markedly slower with silicic acid than with quartz. It was assumed, in an attempt to explain this phenomenon, that the small residue of unreacted silicic acid which persisted in the samples might be a factor by maintaining the silica solution at a high concentration compared to that obtained with quartz. This hypothesis was substantiated qualitatively by increasing the C/S ratio from 0.800 to 0.825 so that excess lime was present to neutralize the excess silica. In that test the transformation to tobermorite was complete in about one day compared to the four to seven days required previously. Direct confirmation of this deduction was indicated later by still unpublished solubility data on the fibrous $C_4S_5H_n$-CSH_n and tobermorite obtained in this laboratory.

Pure tobermorite always appeared in form of thin plates. In one sample, prepared from commercial lime and ground quartz, examination of a stereoscopic electron photomicrograph revealed the presence of a mixture of rolled plates and the usual thin plates. The geometric outline of the rolled crystal plates as they appeared on a photographic print is illustrated in the center of Fig. 7. Generally, these crystals appeared like elongated ellipses with pointed ends. A homogeneous sample of the rolled plates was not knowingly prepared and, therefore, a detailed study of this type of product could not be made. The x-ray pattern of the present sample was identical to that of a well-crystallized sample of tobermorite.

Fig. 7—Electron photomicrograph showing (in center of figure) rolled plate of tobermorite

Results of another series of tests, although obtained on a solid phase deliberately contaminated with Al_2O_3, are presented because the variations in x-ray patterns on portions of this sample stored under different conditions of humidity were considered significant. The sample of CSH_n was prepared

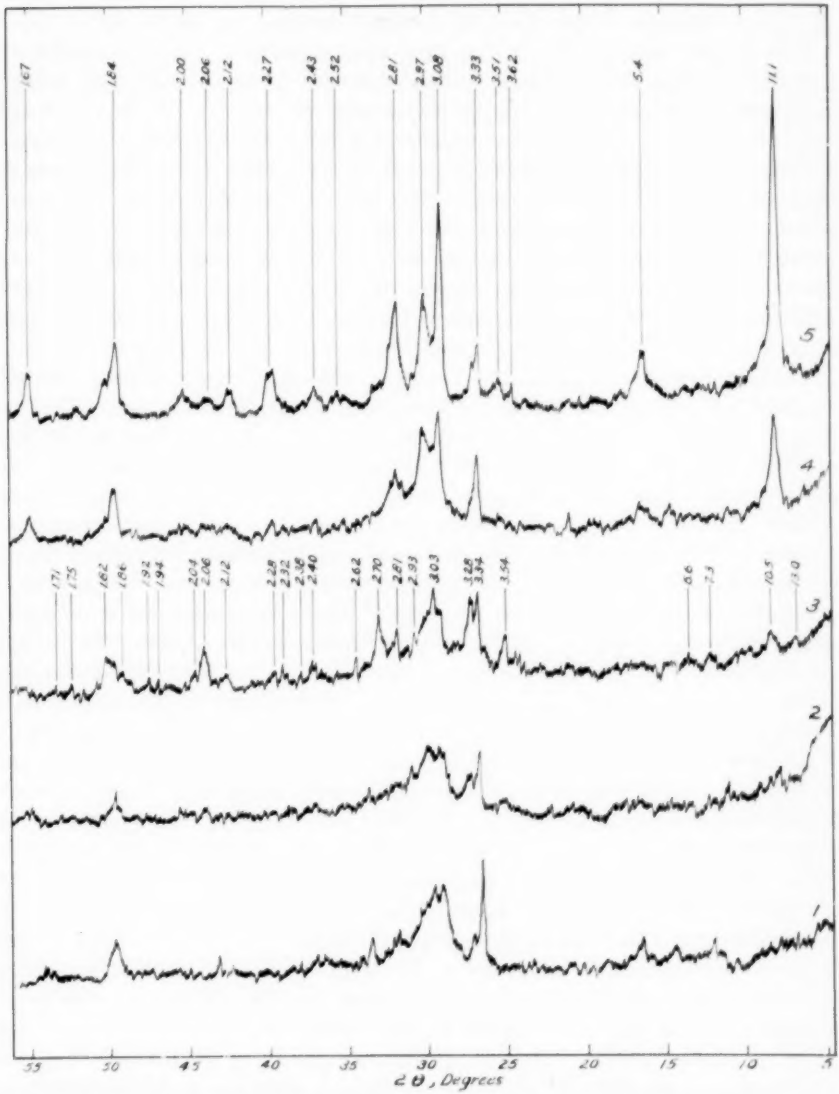


Fig. 8—X-ray patterns of CSH_n dried in vacuum and then exposed to different humidities for three weeks. Patterns are of samples as follows: (1) dried in vacuum four days; (2) dried sample stored at 52 percent relative humidity; (3) dried sample stored at 95 percent relative humidity; (4) dried sample after three-week storage in sealed container; (5) pure synthetic tobermorite

from CaO, ground quartz and 1.9 percent Al_2O_3 as kaolin at 165 C for 15 hr, and was then dried in vacuum for four days. Portions of the sample were stored at 52 and about 95 percent relative humidity (over water in a closed container) for three weeks, and the remainder was set aside in a tightly stoppered bottle. X-ray data of these samples are given in Fig. 8. Two significant changes occurred which are indicated by the x-ray data.

The original sample (No. 1, Fig. 8) showed a pattern similar to the fibrous $C_4S_5H_n$–$C_5S_4H_n$ and apparently is highly similar to that solid phase. However, a few weak lines particularly at spacings of 3.34, 3.21, and 2.65 A correspond to strong lines in the pattern of hillebrandite. This is interesting because hillebrandite, as already mentioned, is one of the three possible lime-rich phases that may form in the initial reactions of lime, silica and water, or portland cement and water at a temperature of approximately 80 to 120 C. One could, therefore, assume that the original sample was a mechanical mixture predominantly of the fibrous CSH_n and a little hillebrandite. This interpretation was not substantiated by x-ray data on the previously dried sample after subsequent dry and damp storage.

The dried sample of CSH_n upon storage in absence of moisture underwent a transformation to the 1.0 C/S tobermorite as suggested by comparison of the pattern (No. 4) of this product to a pattern, No. 5, of pure synthetic tobermorite. The samples rehumidified at 52 and 95 percent as shown by patterns No. 2 and No. 3, respectively, also underwent a change. This change was especially pronounced in the sample stored at 95 percent relative humidity and suggested that the degree of alteration may be related to the amount of water absorbed. For the purpose of comparison with previous data, the spacings of the lines of pattern No. 3 are presented in terms of angstrom units in Fig. 8. This pattern agreed far better with the general x-ray results obtained on hillebrandite and related synthetic products designated as the beta-type hydrate (in Table 1, Reference 4) than with that of any other known hydrous calcium silicate. On the basis of this agreement, it is assumed that the sample has a structure similar to those solids and may possibly be a member of such series of solids previously described.[4] Because of the indicated agreement in x-ray data, the transformed solid, for want of a better name, is designated as the 1.0 C/S beta-type hydrate.

The above results are unusual and, therefore, other similar tests were carried out. The mixes were prepared from freshly slaked lime suspensions, Georgia kaolin, quartz passing the No. 325 screen and with water to solids ratios of 3.0 by weight. The silica of the kaolin was counted as a part of the total, the over-all C/S ratio being 1.0 and Al_2O_3 added amounted to 1, 2, and 3 percent by total weight of dry solids. Autoclaving at 175 C for 16 hr yielded a product containing the principal lines, particularly the line at 11.1 A, of tobermorite. This solid did not undergo any significant transformation in 15 days of moist-air storage (storage over water in a closed system protected against CO_2) following seven days of vacuum drying. The next series was autoclaved at 165 C for 16 hr, and the 2 percent Al_2O_3 sample

also for 21 hr. The products were divided into two portions, one was dried in vacuum and the other in a laboratory oven at 100 C. The 16-hr samples showed patterns more like the fibrous CSH_n but also contained in weak intensity the line at 11.1 A. A marked alteration to the beta-type hydrate was observed after 17 days storage over water. The variations in amounts of Al_2O_3 in these samples had no definite effect on the transformation. The transformation of the 16-hr sample was only slightly more advanced than that of the 21-hr sample. The oven dried sample generally was slower to transform to the beta-type hydrate. Although the transformation was apparently reproduced, no two patterns agreed completely with each other except that the lines were at spacings corresponding to those of the beta-type hydrates. In no sample was a complete pattern of the beta-type hydrate obtained. Only one exception, as follows, was observed in these transformations. The 16-hr sample autoclaved at 165 C, containing 2 percent Al_2O_3 and dried at 100 C, manifested in reduced intensities the principal lines of the alpha-type hydrate after three days storage over water, after nine days the lines were nearly extinct, and at 30 days only lines corresponding to the beta-type hydrate were present.

Transformation of the dried sample to tobermorite in dry storage was not satisfactorily reproduced. It is not clear if the samples after the original drying differed from that used in the first test (Fig. 8). None of the later samples after the first drying and prior to moist-air storage contained the lines of hillebrandite as did the first sample.

Although the transformation of the CSH_n to a beta-type solid apparently occurred due to adsorption of water, attempts to revert the solid to the original by prolonged in-vacuum drying and oven drying at 100 C were unsuccessful. Drying had the effect of sharpening the patterns and changing slightly the intensities of a few of the lines. Resaturation of the completely dried products was also without effect.

The drying-rehumidification tests were repeated with three different samples of pure synthetic tobermorite and the fibrous CSH_n made with silicic acid. The CSH_n remained completely unaltered in all tests. Two of the dried tobermorite samples, upon rehumidification in moist air, manifested a small decrease in the intensity of the 1.84 A line, a small increase in the intensity of the line at 1.82 A, and appearance of a very weak line at 1.88 A. These changes were not detectable in the third sample tested. It appears, therefore, that a dried sample of well crystallized pure tobermorite may undergo a structural change upon adsorption of water, but the change is small and, therefore, difficult to detect. The line at 11.1 A remained completely unaltered for pure tobermorite and related samples ranging in moisture contents in equilibrium with air at 0.0 to 95 percent relative humidity.

The deductions by Bernal, Jeffery and Taylor[9] that volume change of calcium silicate hydrates, as suggested by changes in the spacing and intensity of the line at 11.1 A, may be associated with movement of structural (interlayer) water similarly as in clays is not substantiated directly. The

present observations do not, however, preclude the movement of interlayer water but indicate that such movement is not necessarily detectable by variations of intensity or spacing of the 11.1 A line. Only at moderately elevated temperatures in tests of the present study were the spacing and intensity of the 11.1 A line observed to change.

On the basis of tests on the specimens of fibrous CSH_n without added Al_2O_3 it appears that the substitution of the Al^{3+} in this phase is a factor in contributing to the instability of the structure when exposed to moist air. The tobermorite appeared stable with or without added Al_2O_3. Although results are inconclusive for summary statement regarding the transformation to the beta-type hydrate, two factors are indicated as being significant. First, the adsorption of water produces definite effects which are not reversible, and second, the Al^{3+} appears to be a necessary constituent of the phase in order that transformation occurs. Whether the observed trends would be altered in prolonged tests cannot be predicted, and long-time tests on the phases in question are needed for that reason.

INTERPRETATION OF RESULTS

The possibility that the fibers are actually tightly rolled plates would afford the simplest explanation for the occurrence of the two forms of the solid. This would imply that the difference between the two forms is merely one of geometric shape. If this were true, the two solids should manifest some similarity in physical properties. Results of test, however, show a striking difference. The fibrous solids do not integrate at all, or only slightly, and the wet mass shrinks in the order of 30 to 70 percent during drying. The platy phase, on the other hand, forms a dimensionally stable solid of superior strength.

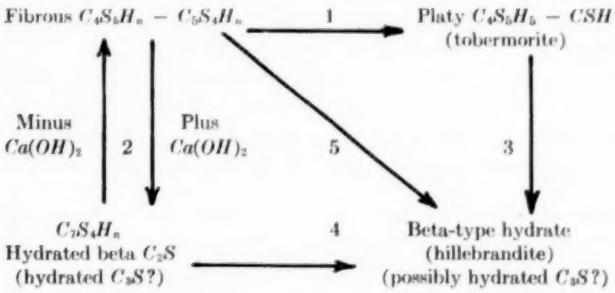
Heller and Taylor[10] prepared hydrothermally a monocalcium silicate hydrate in well crystallized form (which later was shown to be tobermorite[11]) and considered this solid to be the same as that (CSH_n–$C_3S_2H_n$) previously prepared by Taylor[7] at room temperature. Small differences in x-ray data between the two preparations were attributed to inaccuracies in the earlier measurements. Careful measurements on the same types of phases in the present studies show that the differences, although small for most lines of the spectrum, are in fact real. The present results on the fibrous $C_4S_5H_n$–$C_5S_4H_n$ are in excellent agreement with the results on the solids prepared by Taylor at room temperature. Therefore, it is concluded that the x-ray data, indicating presence of two similar solids, fibrous $C_4S_5H_n$–$C_5S_4H_n$ and tobermorite, are in accord with the results of other tests described in this report.

The chemistry of fibrous solids of definite compositions, $C_4S_5H_n$, CSH_n, and $C_5S_4H_n$ has been discussed in some detail in this report. From the standpoint of strict definition, these solids cannot be regarded as distinct phases but rather as a single phase of variable lime content. The same statement applies to the tobermorite series. From a practical consideration, compositions within the given range have to be controlled and therefore an under-

standing of the chemistry of these solids of different compositions is pertinent to a sound technology of hydrous calcium silicates.

On basis of x-ray data alone it could be said, but not with complete certainty, that the $C_7S_4H_n$ is also a member of the above series. The x-ray patterns of all the compositions of the fibrous solids and the $C_7S_4H_n$, unfortunately, consist of only few lines and even these are known to have shown variations among different preparations. For this reason, caution has to be observed in interpreting these results. Significant chemical differences[1] are obtained between the $C_7S_4H_n$ and the $C_4S_5H_n$–$C_5S_4H_n$ and, in view of these, the x-ray data become even less convincing regarding the supposition that one phase represents the entire range of composition. The evidence, however, appears inconclusive for either view and further research on the solids will be required before any definite conclusion can be reached.

From the standpoint of technology the low- and high-lime products have to be regarded as different solids. To facilitate the discussion of the science and technology of these and related solids, the following schematic diagram is presented in terms of reactions even though some of these are not definitely established experimentally.



The fibrous $C_4S_5H_n$–$C_5S_4H_n$, if free of contaminants, is important principally as it applies to the science. This solid, however, may be present with or without $C_7S_4H_n$ in improperly cured sand-lime brick. Tobermorite, on the other hand, is technologically of interest because it probably is the binder of properly cured sand-lime brick. The gel-like $C_7S_4H_n$ which may also occur in the fibrous habit, but not definitely proven to be so, as it tends to agglomerate into rounded grains, is obviously of interest to the chemistry of concrete cured at ordinary temperature.

Beta dicalcium silicate hydrated at room temperature, as shown by Taylor[7] and also observed in the present studies, is probably also $C_7S_4H_n$. Theoretical considerations indicate that pure C_3S should also form this hydrate at room temperature. Taylor, however, called attention to some difference in x-ray diffraction of the hydrates of beta C_2S and C_3S and suggested some structural difference may obtain between the two. In this study a paste of

*Tobermorite may also contain lime in amounts up to 1.25 C/S, but the solid of this composition, and completely free of related phases, has not been synthesized.

C_3S was ground after a hydration period of 28 days, mixed with additional water, and examined by x-ray at the end of 60 days of total time of hydration. The pattern differed from that of Taylor in that it contained a few diffused lines, extraneous of those of $C_7S_5H_n$, suggestive of the beta-type hydrate. The sample of C_3S used, however, contained 1.3 percent Al_2O_3, as a contaminant from the ball mill during grinding between burns, and the results are not directly comparable to those of the pure ternary compounds. Alumina added as kaolin in amounts as low as 0.4 percent Al_2O_3 enhanced significantly the hydrothermal formation of tobermorite, the fibrous $C_4S_5H_n$ being the solid obtained without the added Al_2O_3. The possibility that Al_2O_3 may similarly affect the type of product from C_3S and water should not be overlooked.

The fourth product in the schematic diagram given as beta-type hydrate (synthetic hillebrandite) was inserted more for purposes of discussing the possible products of cement than as a proven product of hydration. The effects of Al_2O_3 (and Fe_2O_3) on the $C_7S_4H_n$ may require detailed study before the binder of concrete can be defined exactly. The reactions or transformation among the different solids in the diagram altered by additions of Al_2O_3 are No. 1, 4, and 5. (Reaction 2 involving adsorption or extraction of lime has not been studied in presence of Al_2O_3, and the possible transformation 3 has not been observed to occur in presence or absence of Al_2O_3.) The enhancement of the formation of tobermorite by addition of Al_2O_3 has already been mentioned. Autoclaved concrete block made according to standard industrial processes may generally be expected to contain tobermorite[12] even in presence of relatively large amounts of contaminants.

The hydration of beta C_2S and C_3S with added Al_2O_3 has not been studied except in the isolated test mentioned. Tests on portland cement, which, of course, contains the ferrite phase and minor constituents, in addition to the three phases of interest beta C_2S, C_3S and C_3A, were carried out as follows to ascertain if the beta-type hydrate is a possible solid in concrete block. A portland cement without added siliceous material was hydrated following a plant cycle with a maximum temperature of 80 C. The product examined by x-ray appeared to consist largely of synthetic hillebrandite. Oil-well cement hydrated at 120 C for 16 hr also contained the beta-type hydrate as the most abundant phase. In other tests following a plant cycle of 80 C as maximum temperature, 40 percent of the cement was replaced by either diatomite or silica gel. The product as indicated by x-ray and DTA results consisted of the fibrous CSH_n–$C_5S_4H_n$ and probably $C_7S_4H_n$. Substitution of either 40 percent tripoli or quartz (passing No. 325 screen) for the cement resulted in possibly a mixture of at least two or possibly three hydrates ($C_7S_4H_n$, beta- and alpha-type hydrates). The x-ray patterns, however, containing lines of $Ca(OH)_2$, quartz and anhydrous beta C_2S, were too complex for the purpose of identifying definitely the products formed. The other observed occurrence of the beta-type hydrate is that according to the

transformation (No. 3 in the diagram) of the fibrous CSH_n upon drying and rehumidification.

Hydrothermal treatment of the $C_7S_4H_n$ or cement pastes at temperatures above approximately 110-120 C, depending on time of processing, results in the formation of the alpha-type hydrate[8] which apparently is noncementitious. If the raw concrete block mix does not contain a sufficient amount of reactive siliceous material and is autoclaved, the alpha-type hydrate forms. It must be emphasized that properly designed and autoclaved concrete products do not contain any of the lime-rich phases mentioned.

To illustrate how basic data, such as that presented in this report, can be applied for explaining the behavior of concrete, the problem of irreversible drying shrinkage of concrete at ordinary temperature is considered. It is well known that autoclaved concrete block dried and then resaturated at room temperature re-expands to the original volume. This is not true of moist-air or low-pressure steam cured products; during the first drying and resaturation a permanent decrease in volume is always observed. The complete reversibility of the autoclaved block for the stated conditions may be expected because it contains tobermorite as binder which showed no structural changes during drying and rehumidification tests. The block cured by the other methods probably contained at least some of the fibrous phase and the $C_7S_4H_n$. The instability of the fibrous phase, and possibly of the $C_7S_4H_n$ because of its structural similarity to that phase, is indicated by the conversion to the beta-type hydrate which, when once formed, does not revert to the original solid. Obviously, the structural change in going from the initial to the transformation solid would have to be a volume decrease. The substantiation or invalidation of this hypothesis is amenable to direct experimental test.

Application of the hypothesis in a recent report[13] that movement of "interlayer" water is the cause of shrinkage or swelling of concrete (exclusive of the permanent initial shrinkage of normally cured concrete) is indicated by the x-ray results for dried and rehumidified tobermorite even though these were small. Further similar studies on tobermorite, not the pure phase, but as it forms in autoclaved concrete products, should prove worthwhile.

FACTORS AFFECTING REPRODUCIBILITY OF RESULTS IN THE SYSTEM C-S-H

In phase-equilibria studies carried out for the sole purpose of isolating equilibrium products, one may disregard several variables in the procedures without producing any ill effects on the final result. Among these factors are the physical properties of the raw materials such as fineness, temperatures, and times of processing within wide ranges, amount of mixing water in excess of that required to form a given hydrate, and several others to be mentioned. In studies, such as the present one, concerned primarily with the chemistry of intermediate phases this no longer holds true. Careful control of all factors must be maintained if reproducible results are to be obtained. The importance of this cannot be overemphasized.

Some of the difficulties experienced in the earlier phases of the present study and in related studies[1,4,5,8,14] are briefly reviewed. It seemed that the discussion might be helpful to readers in reconciling apparently contradictory results from different laboratories and perhaps be of aid to future research workers in setting up working procedures in studies of these solids.

The effects caused by substituting silicic acid for quartz, all other factors being held constant, is apparent from results of this report. Also, it is shown that identical products are obtained from the two forms of silica if the proper processing conditions are selected for each form of silica. In synthesis[4] of lime-rich crystalline calcium silicate hydrates the form of silica appears also to be important regarding the type of solid obtained. In other instances, such as the synthesis of xonotlite at temperatures above about 175 C, silicic acid and quartz may be used interchangeably. Tripoli, a microcrystalline quartz of high purity (99.2 percent SiO_2) was used successfully in the synthesis of tobermorite of 1.0 C/S ratio (175 C for 8 hr). In a parallel experiment with quartz fines (passing a No. 325 screen) the product formed was a mixture of the fibrous phase, tobermorite, and some xonotlite. The choice of silica, not only as quartz or silicic acid, but even among different brands of silicic acid or silica gel, may have an effect on the kind of intermediate product formed. In the preparation of the lime-rich precursor of the alpha-type hydrate,[8] a change from one brand of silicic acid to another altered markedly the rates of reaction, and until this difficulty was recognized results of earlier tests could not be reproduced.

Unpublished data of a still incomplete project in this laboratory on the compositions of solutions required to stabilize the various low-lime hydrous calcium silicates show that gyrolite required a comparatively high silica concentration for stabilization and this explained why silicic acid, instead of quartz which is of low solubility, is required for synthesizing this particular phase. Parenthetically, it has to be stated that completely pure gyrolite was not obtained; the solutions high in silica also stabilized the fibrous $C_4S_5H_n$–$C_4S_5H_n$ and this phase persisted (as shown by electron microscopy, x-ray, and DTA results) in small amounts even to the highest temperature used, 300 C. It was not ascertained if a lower concentration of silica was required for preparing gyrolite than that for the fibrous phase. In any case, however, it appears that the concentration of silica in solution is a determining factor in the type of product that forms.

Calcium hydroxide occurs only in one crystalline form and therefore only the physical state of this solid has to be controlled. In a previous study[14] it was observed that $Ca(OH)_2$ in crystals about 10 to 70 microns in diameter became embedded in the reaction product, a hydrous lime-soda silica solid, formed during the early stages of the reaction and thereafter reacted very slowly. Substitution of finely powdered $Ca(OH)_2$ in duplicate tests resulted in a marked acceleration of the rate of reaction. Although $Ca(OH)_2$ was not used in large crystals in the present studies, it was observed that previously hydrated and dried lime containing agglomerates of the solid up to a size

just passing a No. 20 screen did not react as rapidly as did freshly slaked lime in water suspensions. Also, homogeneous solids were not as easily obtained with the granular dried $Ca(OH)_2$ as with that in water suspensions. The best method of introducing the lime appeared to be in the form of a dispersed suspension of freshly slaked lime. The quicklime so used must not be over-burned and should be of such reactivity as to hydrate completely in about 2 to 4 min at a temperature of about 95-100 C, starting with water at about 50 C. Six parts of water to one part of properly burned CaO of reagent quality by weight are required for forming a slurry of heavy cream consistency completely free of any visible particles.

If water is added to a mix in an amount sufficient to form a particular hydrate, it would be expected that variation in amount of any excess water would have no significant effect on the results. Contrary to this belief results of tests showed that marked changes are caused in the reaction product by decreasing the water content from a water to solids ratio of 3.0 to 0.8 by weight (W/S ratio) in mixes made with finely subdivided $Ca(OH)_2$ and finely ground quartz. The pastes of the low water content having a composition from 0.8 to 1.2 C/S and autoclaved at 175 C yielded products which tended to undergo the exothermic reaction in DTA at a fixed temperature. Similar results on cement-quartz mixes (W/S of 0.75) have been published.[8] The solids in this series of tests manifested small DTA effects suggestive of the presence of a lime-rich phase.[12] These results indicate the application of DTA for testing the homogeneity of the reaction product. As noted previously,[1] if the fibrous calcium silicates are homogeneous, each composition manifests a characteristic exothermic effect. Highly watered suspensions of W/S ratios of 6:1 and 3:1 are used in commercial production of hydrous calcium silicates, whereas concrete and concrete products are made with mixes of much lower water contents. The tendency for the presence of nonhomogeneous solids in autoclaved concrete products is more pronounced than in the lightweight hydrous calcium silicate. However, the time of processing is a factor; if given time, the products in the low-water content mix should react to a stage obtained with the highly watered mixes.

Factors besides those discussed above are the methods of mixing mentioned in a previous report,[4] possibly the type of storage following the first drying, and the rate of up-pressure in the autoclaving cycle. The method of preparing pastes or suspensions of the beta C_2S and C_3S and subsequent treatment of these such as grinding to accelerate the reaction would be expected to have effects on the products obtained.

Some of the causes of nonreproducibility of results caused by a change in one factor can be explained, such as formation of gyrolite from silicic acid and not from quartz. No satisfactory explanation is apparent for others. One may, however, speculate that the reactivity of large crystals or granules of minute crystals of $Ca(OH)_2$ is due to the formation of a slightly pervious coating on the surface of the solid during the earlier stages of the reaction. The minute crystals of $Ca(OH)_2$ on the other hand, react completely before

the coating reaches a thickness great enough to become slightly pervious. Changes in products caused by decreasing the amount of water in mixes may be due to a similar effect; the reaction products having less intra-particle space in which to precipitate form denser coatings on the still incompletely reacted product of reaction. So long as free $Ca(OH)_2$ is present the initial reaction product is the $C_7S_4H_n$ or related solid. It could therefore be expected that the solid formed from the last amount of $Ca(OH)_2$ in the core of the surrounding shell of hydrated product would be a lime-rich phase. This product forming the core, depending on the permeability of the surrounding mass to the diffusion of the ions, might react slowly with the residual silica which itself may be similarly surrounded with hydration products.

SUMMARY

1. Hydrothermal reaction products of $Ca(OH)_2$ and silicic acid prepared under certain conditions at temperatures of 125 to 175 C are fibrous solids extending in composition from $C_4S_5H_n$ to $C_5S_4H_n$.

2. The compositions of $C_4S_5H_n$, CSH_n, and $C_5S_4H_n$ were differentiated by DTA. All, however, showed the same x-ray pattern.

3. Upon prolonging the autoclaving, the fibrous $C_4S_5H_n$ transformed slowly to the mineral tobermorite, $C_4S_5H_5$, a platy crystalline solid.

4. The same fibrous products were also obtained by using pulverized quartz but the reactions were much slower. The final transformation of $C_4S_5H_n$ to tobermorite, however, was comparatively rapid. This difference in rate was attributed to the difference in the concentration of silica in the aqueous phase.

5. The initial products in reactions between $Ca(OH)_2$ and different forms of silica are lime-rich (1.75 C/S) phases. Three different types are described and the raw material from which they may form are given.

6. The reactions between $Ca(OH)_2$ and different forms of silica occur in steps, the first product always being a lime-rich phase. The subsequent products, as more and more silica reacts, for a 0.8 C/S mix, in order of formation, are as follows: $C_5S_4H_n$, CSH_n, and $C_4S_5H_n$, and finally the latter transforms to tobermorite.

7. Results of studies with x-ray, DTA, electron microscopy, BET nitrogen adsorption, and chemical analysis show that two types of similar low-lime products have been synthesized; in form, one is fibrous and the other platy.

8. The fibrous products do not integrate and the resulting mass upon drying undergoes a marked shrinkage. The platy phase, however, formed into a dimensionally stable solid of superior strength.

ACKNOWLEDGMENTS

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Title No. 51-51

Stability of Reinforced Concrete Retaining Walls and Abutments*

By W. REJMAN†

SYNOPSIS

An analytical method is presented whereby minimum width of horizontal base and its relative position can be found for any shape or loading of a cantilever or counterforted retaining wall. The method is particularly applicable to loaded or unorthodox retaining walls such as in basements of some industrial buildings, ditch bunkers, and abutments.

INTRODUCTION

Design of standard cantilever and counterforted retaining walls whose only function is to resist pressure of the retained material is assisted by "direct" formulas or charts from which approximate proportions of the wall can be found. This is followed by detailed investigation of stability. Adjustments are made and investigation repeated if the first assumption is not satisfactory.

Stability of retaining walls carrying loads, or those of unusual shape, is investigated by the same procedure of selection and trial but no general formulas exist or can be developed to assist in the first assumption. It is in this type of retaining wall that determination of the base is particularly difficult. The procedure involves various corrections to an assumed base, each followed by an analytical or graphical examination of stability. With complicated loading, the number of such operations may be considerable until a final solution is reached. Even then it is not always certain if the design is most economical.

There are two unknown factors with regard to the base of a retaining wall—the width and relative position to the wall proper—with an unlimited number of combinations of the two. Theoretically, only one such combination will satisfy all conditions of stability with maximum economy. The proposed method reduces the choice of the base to its width only. The other unknown factor, its relative position, is found. The tedious work just mentioned is thus eliminated and the problem solved with two or three assumptions of the width followed by simple computations.

The method is not approximate. In all its operations actual loads are introduced and the results are final.

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NOTATIONS, DEFINITIONS, AND SIGN CONVENTION

a, b, c, d = coefficients in equations for M and N

e = eccentricity of resultant force at lower face of base ($e = M/N$)

h = height from top of base to top of backfill (or to top of surcharge if one is assumed)

H = total horizontal load on retaining wall

heel = back portion of base cut off by plane Y

l = width of base

μ = coefficient of friction between base and soil

M = total moment about center of lower face of base ($M = Ne$)

M_o = moment of N_o, or its components, about plane Y

M_H = moment of H, or its components, about lower face of base

n_o, n_s = specified factors of safety (overturning, sliding); also actual coefficients of safety

N = total vertical load

N_o = $N - (W_B + W_E)$

p = specified allowable ground pressure; also actual maximum ground pressure

r = specified limit of ratio e/l; also actual e/l

R = passive resistance of earth in front of wall

w = unit weight of earth in backfill

W_B = weight of base

W_E = weight of earth above the heel ($W_E = whx$)

x = width of heel

Y = vertical plane tangent to the back face of wall proper

Moments consistent with overturning moment are positive.

Moments opposite to overturning moment are negative.

Loads and moments are per foot of wall.

THE CONCEPT AND BASIC EQUATIONS

Examples of various types of retaining walls under consideration are shown in Fig. 1. They are all composed of the wall proper, the counterforts (if any), and the base. The wall proper may be vertical or inclined, straight, broken or curved, and may act as a cantilever or be supported by counterforts. The base is horizontal and of rectangular section.

Loads acting on a retaining wall are shown in Fig. 2. They can be divided into three groups:

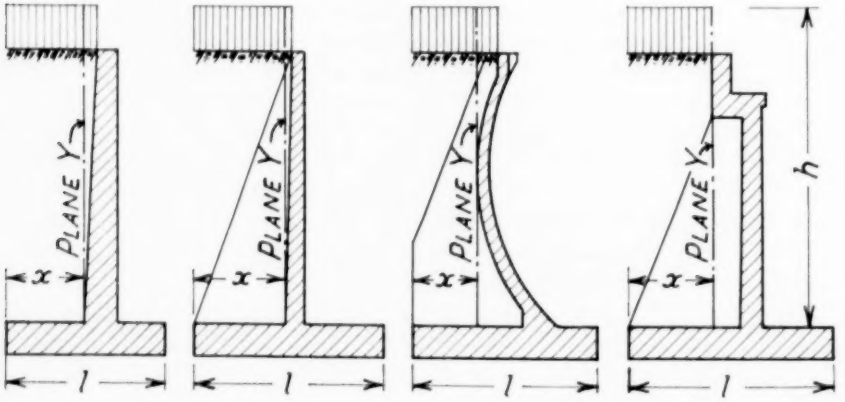


Fig. 1—Position of plane Y in various types of retaining walls

Loads independent of l and x—These are weights W_1, W_2, W_3, etc., of all concrete and earth within the space enclosed by the top of the base, external outline of the wall proper, and plane Y;* earth pressure P; and loads L_1, L_2, L_3, etc., (vertical or horizontal) acting on the wall proper or on the toe. All these loads are known. Their magnitude and position determine the values of N_o, H, M_o, and M_H.

Load dependent on l—Weight of the base, W_B.

Load dependent on x—Weight of earth above the heel, $W_E = whx$.

Let the position of the base in relation to the wall proper be variable. This can be expressed by the variable width of the heel, x. Assuming that the width and weight of the base are known, total moment about center of base and total vertical load can be expressed as functions of x:

$$M = M_o + N_o(x - l/2) + M_H - whx(l - x)/2$$

$$M = \frac{wh}{2}x^2 + \left(N_o - \frac{wh}{2}l\right)x + (M_o + M_H - N_o l/2) \quad \text{or}$$

$$M = ax^2 + bx + c \qquad (1)$$

where

$$\left.\begin{aligned} a &= wh/2 \\ b &= N_o - al \\ c &= (M_o + M_H) - N_o l/2 \end{aligned}\right\} \qquad (1a)$$

$$N = whx + N_o + W_B \quad \text{or}$$

$$N = 2ax + d \qquad (2)$$

where a is the same as in Eq. (1) and

$$d = N_o + W_B \qquad (2a)$$

It will be noticed that a is independent of l, and b, c and d can promptly be modified if the width of the base is altered. All the coefficients must be given proper signs resulting from the equations.†

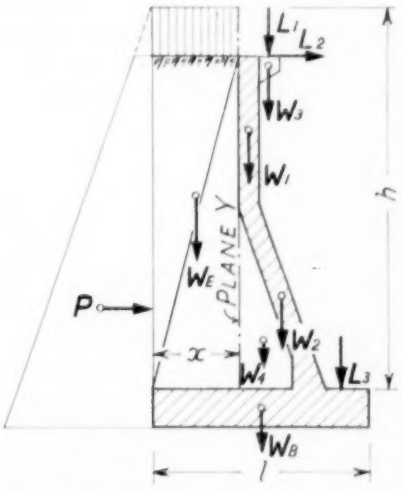


Fig. 2—Loads acting on a retaining wall

*The weight of counterforts over that of earth in their place is neglected as in conventional investigation. It may be introduced, however, if desired.

†a and d are always positive, b is usually negative unless the wall is heavily loaded.

With Eq. (1) and (2) established, a relation can now be found between the position of the base and each condition of stability governing the design of retaining walls.

Distribution of ground pressure

It is generally required that ground pressure should extend over the whole area of the base, *i.e.*, $e/l \leqq 0.167$.* For plastic or yielding soils the limit is further restricted to prevent excessive differential settlement and tilting of the wall. The condition in general form is $e/l \leqq r$ or $M/Nl \leqq r$. Substituting Eq. (1) and (2) for M and N, it becomes:

$$ax^2 + (b - 2alr)\,x + (c - dlr) \leqq 0 \qquad (3)$$

If p and p' are maximum and minimum pressures under the base, then $p'/p = (1-6r)/(1+6r)$. The relation is shown in Table 1 from which proper values of r can be selected.

TABLE 1—VALUES OF r AND p'/p

$r = e/l$	0	0.02	0.04	0.06	0.08	0.10	0.12	0.14	0.16	0.167
p'/p	1.00	0.78	0.61	0.47	0.35	0.25	0.16	0.09	0.02	0

Safe ground pressure

Maximum ground pressure under the base must not exceed the specified value of p, $N/l + 6M/l^2 \leqq p$. Substituting Eq. (1) and (2) for M and N, the condition becomes:

$$ax^2 + \left(b + \frac{al}{3}\right)x + \left[c + \frac{l}{6}(d - lp)\right] \leqq 0 \qquad (4)$$

Sliding

A retaining wall which resists sliding by friction between the base and the soil and by passive resistance of earth in front of it must carry such a load on the foundation that condition $\mu N + R \geqq n_s H$ is satisfied. Substituting Eq. (2) for N, the condition becomes:

$$x \geq \frac{\dfrac{n_s H - R}{\mu} - d}{2a} \qquad (5)$$

Since each of the factors on the right-hand side of the inequality is positive, no heel is required when $d \geqq (n_s H - R)/\mu$. This may be the case of a heavy permanent load acting on the wall. Usually, retaining walls require extra earth load on the heel for their stability against sliding, especially when passive resistance does not exist.

Overturning

If a retaining wall is founded on rock, safety against failure by overturning is secured when $-M_N/M_H \geqq n_o$, where M_N is moment of all vertical loads about the toe. Substituting $M = Nl/2 - M_H$ for M_N and Eq. (1) and (2) for M and N, the condition becomes:

*See "Concluding Remarks" for treatment of cases where uplift of the heel is permitted.

$$ax^2 + (b - al)\,x + \left[c - \frac{dl}{2} + (n_o - 1)\,M_H\right] \leq 0 \quad \ldots\ldots (6)$$

Eq. (3), (4), (5), and (6) represent the four basic conditions of stability. Each equation sets a limit or limits to the position of the assumed base. Sliding is prevented when sufficient heel is provided as computed from Eq. (5). Other requirements are satisfied when the heel is maintained within limits found from Eq. (3), (4), or (6). Since a is positive, the last three inequalities are fulfilled when x lies between roots of the corresponding quadratic equations.* These roots, as well as those of Eq. (7) below, can be found from Table 3 and Eq. (c) in the appendix.

The most favorable position of the base from the angle of pressure distribution occurs when $e = M/N$ = minimum. Substituting Eq. (1) and (2) for M and N and putting $\partial e/\partial x = 0$, the condition of minimum eccentricity becomes:

$$ax^2 + dx + \left(\frac{bd}{2a} - c\right) = 0 \quad \ldots\ldots (7)$$

Eq. (7) does not represent any particular condition of stability but may be useful in determining final position of the base already selected.

DESIGN

Application of basic equations to design

The proposed method is a selection and trial procedure in which the width of the base is assumed and its relative position found from Eq. (3) to (6). The problem is solved when all conditions of stability are met simultaneously with minimum width of the base, *i.e.*, when the resultant limits of x obtained from Eq. (3) to (6) are real, feasible, and close. The width must be increased when any particular limit is imaginary or when the limits are real but conflicting. On the other hand, the width must be reduced when the resultant limits are too wide apart. An "ideal" assumption of the base is when the resultant limits coincide,† as in Example 2, but it should be considered satisfactory if they are 1 or 2 ft apart. Final position of the base can then be chosen at any intermediate point. Eq. (7) can be used to help in the choice but it is only in certain cases that the position of minimum eccentricity comes within final limits obtained from conditions of stability.

In practice the number of conditions to be considered can be reduced. Eq. (6) should be used only for walls founded on rock. Where a retaining wall is prevented from sliding by a structure or floor in front of it, or is carrying a heavy permanent load, Eq. (5) can also be omitted. Only two equations, Eq. (4) and (5) for poor soils or Eq. (3) and (5) for firmer soils, can be used in cases of nonloaded or lightly loaded retaining walls which resist sliding by friction, possibly assisted by a limited passive resistance of earth.

*When one root is positive and another negative, there is only one limit, $x \leq$ positive root.
†See definition of minimum width in "Concluding Remarks."

Checking equations

After the widths of the base and the heel have been found, values of M and N are computed from Eq. (1) and (2) and stability of the structure checked with the following equations:

$$r = \frac{M}{Nl} \quad (8)$$

$$p = \frac{N}{l}(1 + 6r) \quad (9)$$

$$n_s = \frac{\mu N + R}{H} \quad (10)$$

$$n_o = 1 + \frac{\frac{Nl}{2} - M}{M_H} \quad (11)$$

Procedure

The computation is made per foot of wall and the procedure is as follows:

(a) Design the wall proper.

(b) Assume or compure the depth of the base. In cantilever walls it can be assumed equal to the thickness of the wall at the bottom. In counterforted walls it can be computed for load wh, reduced by p' from Table 1, as a continuous slab supported by counterforts.* Generally, small variations in depth of the base have negligible effect on final results.

(c) Find loads independent of l and x as specified in the preceding paragraph.

(d) Compute N_o, H, M_o, M_H, R, and $a = wh/2$.

(e) Assume l and compute W_B.

(f) Find coefficients b, c, and d from Eq. (1a) and (2a).

(g) Find limits of x for selected conditions of stability as outlined above. For sliding use Eq. (5) directly, for other conditions use Table 3 and Eq. (c) in the appendix.

(h) If the result is not satisfactory, change l and repeat operations (e) to (g) until the resultant limits are close enough.

(i) For final l and x compute M and N from Eq. (1) and (2) and check stability with Eq. (8) to (11).

Where complex dead, live, traffic, crane, or wind loading occurs, two or more combined loads must be selected and the procedure carried out simultaneously for each load. Sliding should be investigated under dead load only. Consideration should also be given to the sequence of loads applied, *e.g.*, whether backfilling is to precede or to follow erection of the superstructure.

EXAMPLES

Example 1

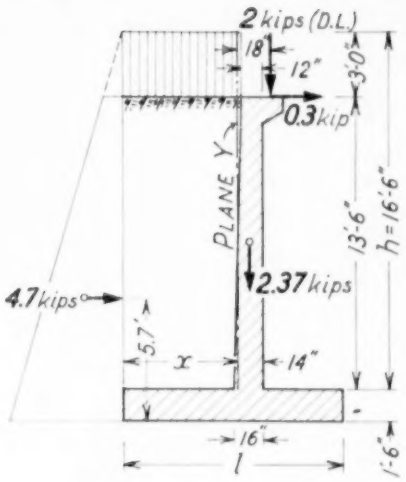
Find the base of a retaining wall founded on a poor soil under the following assumptions: $p = 1.8$ kips per sq ft, $r = 0.1$, $n_o = 1.5$, $w = 0.1$ kip per cu ft. Dimensions of the wall proper, depth of the base, and loads are shown in Fig. 3. The superstructure is to be erected and a concrete floor in front of the base cast prior to backfilling. The wall is thus prevented from sliding by the floor.

For simplicity the trapezoidal section of the wall proper is replaced by an equivalent rectangle. Starting from point (d) of the "Procedure:"

$N_o = 2.37 + 2.00 = 4.37$ kips $\qquad M_o = 2 \times 1.5 + 2.37 \times 0.583 = 4.4$ kip-ft

*For the fixed end of the toe the depth can be increased later, if required, by providing a slope.

Fig. 3—Retaining wall in Examples 1–3



$H = 4.70 + 0.30 = 5.00$ kips $\qquad M_H = 4.7 \times 5.7 + 0.3 \times 15 = 31.3$ kip-ft

$a = 0.1 \times 16.5/2 = 0.825$ $\qquad 1.a = 1.22$ $\qquad M_v + M_H = 35.7$ kip-ft

First assumption: $l = 9$ ft

$W_B = 1.5 \times 9 \times 0.15 = 2.03$ kips $\qquad c = 35.7 - 4.37 \times 9/2 = 16.0$

$b = 4.37 - 0.825 \times 9 = -3.05$ $\qquad d = 4.37 + 2.03 = 6.40$

Eq. (4): (see Table 3 in the appendix)

$$\beta = -\frac{3.05}{0.825} + \frac{9}{3} = -0.70 \qquad \gamma = 1.22\left[16 + \frac{9}{6}(6.4 - 9 \times 1.8)\right] = 1.59$$

Since $\left(\frac{\beta}{2}\right)^2 < \gamma$, the limits of x are imaginary and l must be increased.

Second assumption: $l = 10$ ft

$W_B = 1.5 \times 10 \times 0.15 = 2.25$ $\qquad c = 35.7 - 4.37 \times 10/2 = 13.85$

$b = 4.37 - 0.825 \times 10 = -3.88$ $\qquad d = 4.37 + 2.25 = 6.62$

Eq. (4):

$$\beta = -\frac{3.88}{0.825} + \frac{10}{3} = -1.37 \qquad \gamma = 1.22\left[13.85 + \frac{10}{6}(6.62 - 10 \times 1.8)\right] = -6.2$$

$$x_{1,2} = 0.68 \pm \sqrt{0.68^2 + 6.2} \qquad x \leqq 3.26 \text{ ft}$$

Eq. (3):

$$\beta = -\frac{3.88}{0.825} - 2 \times 10 \times 0.1 = -6.7 \qquad \gamma = 1.22\,(13.85 - 6.62 \times 10 \times 0.1) = 8.8$$

$$x_{1,2} = 3.35 \pm \sqrt{3.35^2 - 8.8} \qquad 1.80 \leqq x \leqq 4.90 \text{ ft}$$

The resultant limits are $1.80 \leqq x \leqq 3.26$ ft. Since the limits are only 1.46 ft apart, no reduction of l is practicable. Eq. (7) gives $x = 3.2$ ft which indicates that the best choice of x is near the upper limit.

Checking of stability: assume $l = 10$ ft, $x = 3$ ft

Eq. (1): $M = 0.825 \times 3^2 - 3.88 \times 3 + 13.85 = 9.63$ kip-ft

Eq. (2): $N = 2 \times 0.825 \times 3 + 6.62 = 11.57$ kips

Eq. (8): $r = \dfrac{9.63}{11.57 \times 10} = 0.084 \qquad (< 0.1)$

Eq. (9): $p = \frac{11.57}{10}(1 + 6 \times 0.084) = 1.74$ kip per sq ft (< 1.8)

Eq. (11): $n_o = 1 + \frac{11.57 \times 10/2 - 9.63}{31.3} = 2.54 \quad (> 1.5)$

Example 2

Find the base of a retaining wall as in Example 1 assuming that there is no floor in front of the base and the structure resists sliding by friction ($\mu = 0.4$) and passive resistance of earth $R = 0.5$ kip. Factor of safety against sliding $n_s = 1.5$, other conditions as before.

Values of N_o, H, M_o, M_H, and a are the same as in Example 1. Since sliding is resisted almost entirely by friction, a wider base is required.

First assumption: $l = 11$ ft

$W_B = 1.5 \times 11 \times 0.15 = 2.47$ $\qquad c = 35.7 - 4.37 \times 11/2 = 11.7$

$b = 4.37 - 0.825 \times 11 = -4.70$ $\qquad d = 4.37 + 2.47 = 6.84$

Eq. (4):

$$\beta = -\frac{4.70}{0.825} + \frac{11}{3} = -2.05 \qquad \gamma = 1.22\left[11.7 + \frac{11}{6}(6.84 - 11 \times 1.8)\right] = -14.6$$

$x_{1,2} = 1.03 \pm \sqrt{1.03^2 + 14.6}$ $\qquad x \leqq 5.00$ ft

Eq. (5):

$$x \geqq \frac{\frac{1.5 \times 5 - 0.5}{0.4} - 6.84}{2 \times 0.825} \qquad x \geqq 6.45 \text{ ft}$$

Since limits of x for maximum pressure and sliding are conflicting, a larger base must be assumed. It is evident, however, that only a small increase is necessary.

Second assumption: $l = 12$ ft

$W_B = 1.5 \times 12 \times 0.15 = 2.70$ $\qquad c = 35.7 - 4.37 \times 12/2 = 9.50$

$b = 4.37 - 0.825 \times 12 = -5.53$ $\qquad d = 4.37 + 2.70 = 7.07$

Eq. (4):

$$\beta = -\frac{5.53}{0.825} + \frac{12}{3} = -2.7 \qquad \gamma = 1.22\left[9.5 + \frac{12}{6}(7.07 - 12 \times 1.8)\right] = -23.7$$

$x_{1,2} = 1.35 \pm \sqrt{1.35^2 + 23.7}$ $\qquad x \leqq 6.40$ ft

Eq. (5):

$$x \geqq \frac{\frac{1.5 \times 5 - 0.5}{0.4} - 7.07}{2 \times 0.825} \qquad x \geqq 6.30 \text{ ft}$$

Resultant limits are $6.30 \leqq x \leqq 6.40$ ft. This is the case of an almost "ideal" assumption of l where the resultant limits coincide.

Checking of stability: assume $l = 12$ ft, $x = 6.33$ ft

Eq. (1): $M = 0.825 \times 6.33^2 - 5.53 \times 6.33 + 9.50 = 7.6$ kip-ft

Eq. (2): $N = 2 \times 0.825 \times 6.33 + 7.07 = 17.5$ kips

Eq. (8): $r = \frac{7.6}{17.5 \times 12} = 0.0362 \quad (< 0.1)$

Eq. (9): $p = \frac{17.5}{12}(1 + 6 \times 0.0362) = 1.77$ kip per sq ft (< 1.8)

Eq. (10): $n_s = \frac{0.4 \times 17.5 + 0.5}{5} = 1.50 \quad (= 1.5)$

Eq. (11): $n_o = 1 + \frac{17.5 \times 12/2 - 7.6}{31.3} = 4.10 \quad (> 1.5)$

Note the difference in the width and position of the base in Examples 1 and 2. It would be considerably larger had the wall been not loaded and passive resistance of earth not assumed in Example 2.

In both examples checking for overturning was done only to demonstrate the use of Eq. (11). It can be omitted in all cases where foundation material is other than rock.

If δ_1 and δ_2 represent differences between the upper and lower resultant limits of x for assumed l_1 and l_2, respectively,* l_1 being smaller than l_2, and if $\delta l = l_2 - l_1$, minimum width of the base can be obtained approximately by linear interpolation or extrapolation using Eq. (12).

$$l = l_1 + k\,\delta_l \qquad (12)$$

where $k = \dfrac{\delta_1}{\delta_1 - \delta_2}$

Similarly, if x_1 and x_2 represent *upper* limits of x for l_1 and l_2, respectively, and $\delta_x = x_2 - x_1$, position of the minimum base can be obtained from Eq. (13)

$$x = x_1 + k\,\delta_x \qquad (13)$$

Application of Eq. (12) and (13) is shown in Example 3.

Example 3

Let the second assumption in Example 2 be $l = 13$ ft instead of 12 ft. The resultant limits of x can then be found as being: $6.18 \leqq x \leqq 7.72$ ft.

First assumption: $l_1 = 11$ ft; $\delta_1 = 5.00 - 6.45 = -1.45$ ft; $x_1 = 5.00$ ft

Second assumption: $l_2 = 13$ ft; $\delta_2 = 7.72 - 6.18 = 1.54$ ft; $x_2 = 7.72$ ft; $\delta_l = 13 - 11 = 2$ ft; $\delta_x = 7.72 - 5.00 = 2.72$ ft; $k = \dfrac{-1.45}{-1.45 - 1.54} = 0.485$

Eq. (12): $l = 11.00 + 0.485 \times 2 = 11.97$ ft

Eq. (13): $x = 5.00 + 0.485 \times 2.72 = 6.32$ ft

The figures obtained are in agreement with the result of the second assumption in Example 2.

By using Eq. (12), the number of trials can always be reduced to two provided the assumed l_1 and l_2 are reasonably close to the minimum width and the limits of x are real.† With the same reservations, final computation can be reduced to checking of stability if position of the base is found from Eq. (13).

CONCLUDING REMARKS

1. When applied to standard nonloaded retaining walls, the proposed method eliminates any arbitrary assumptions with regard to the relative position of the base. It can be found that in many cases the often recommended toe-to-base ratio of 1/3 is not justified.

2. Definite economy in excavation and concrete can be obtained by designing nonloaded or lightly loaded retaining walls as prevented from sliding, wherever this is justified by the existence of an adequate lateral support in front of the base at the time of and after backfilling.

*It follows that δ is negative when the limits are conflicting.

†To avoid imaginary limits, it is advisable to start with a base too wide rather than too narrow.

3. Wherever practicable, backfilling of loaded retaining walls or abutments, not prevented from sliding, should be postponed until after erection of the superstructure. With this assumption, a smaller base will result except when heavy live loads occur.

4. As a result of the foregoing discussion, the following definition can be formulated: *Width of the base of a retaining wall or abutment is minimum when resultant limits of the width of its heel coincide.*

Usually each of the two limits is determined by a different condition of stability and the "ideal" base (*i.e.*, the base of minimum width in its right position) satisfies these two conditions at their limits, other conditions being satisfied within their limits. In some cases, however, both the coinciding limits of the heel are determined by the same condition of stability and limit of this condition only is reached with the "ideal" base. It may, therefore, be concluded that no general criterion of minimum base can be based merely on final results of investigation of stability.

5. Where, in cases of very firm ground or rock foundation, an uplift of the heel is permitted ($r > 0.167$), Eq. (4) and (9) for maximum pressure do not apply and the following should be used instead:

$$a\left(4a + \frac{3}{2}p\right)x^2 + \left[4ad + \frac{3}{2}p(b - al)\right]x + \left[d^2 + \frac{3}{2}p\left(e - \frac{dl}{2}\right)\right] \leq 0 \quad \text{(4a)}$$

$$p = \frac{2N}{3l\,(0.5 - r)} \quad \text{(9a)}$$

All other equations and the procedure remain unchanged.

If m is the horizontal base of the triangle of pressure, then $m/l = 3(0.5 - r)$. The relation is shown in Table 2 from which values of r can be selected for use in Eq. (3).

TABLE 2—VALUES OF r AND m/l

$r = e/l$	0.167	0.20	0.25	0.30	0.35	0.40	0.45	0.50
m/l	1.00	0.90	0.75	0.60	0.45	0.30	0.15	0

However, distribution of pressure is not always of importance in such cases and Eq. (3) can often be omitted.

Where the foundation is solid rock and no sliding is possible, the base can be determined with Eq. (4a) and (6) for heavily loaded and Eq. (6) alone for lightly loaded or unloaded retaining walls.

APPENDIX

By dividing each side of the inequalities (3), (4), and (6) by a, they can be reduced to a common form:

$$x^2 + \beta x + \gamma \leq 0 \quad \text{(a)}$$

Similarly, Eq. (7) can be expressed by

$$x^2 + \beta x + \gamma = 0 \quad \text{(b)}$$

In both cases the solution consists in finding roots of an equation of the type of Eq. (b). Coefficients β and γ can be computed from Table 3 and the roots from Eq. (c).

TABLE 3—COEFFICIENTS β AND γ IN EQ. (b)

Condition	Equation	β	γ
Pressure distribution	(3)	$\frac{b}{a} - 2lr$	$\frac{1}{a}(c - dlr)$
Safe pressure	(4)	$\frac{b}{a} + \frac{l}{3}$	$\frac{1}{a}\left[c + \frac{l}{6}(d - lp)\right]$
Overturning	(6)	$\frac{b}{a} - l$	$\frac{1}{a}\left[c - \frac{dl}{2} + (n_o - 1)M_H\right]$
Minimum eccentricity	(7)	$\frac{d}{a}$	$\frac{1}{a}\left(\frac{bd}{2a} - c\right)$

$$x_{1,2} = -\frac{\beta}{2} \pm \sqrt{\left(\frac{\beta}{2}\right)^2 - \gamma} \quad \text{(c)}$$

The roots are imaginary when $\left(\frac{\beta}{2}\right)^2 < \gamma$.

Precast Prestressed Lightweight Concrete Construction*

By ARTHUR M. JAMES†

SYNOPSIS

Design, fabrication, and erection of precast and prestressed floor and roof beams and slabs of lightweight expanded shale concrete are described. A discussion of the elastic modulus of expanded shale concrete is based on measured camber at prestressing. Cost data on the beams and slabs and a yard test of one prestressed beam are included.

INTRODUCTION

The two jobs described are interesting not for their size or complexity, but because they demonstrate that precast prestressed concrete is reasonably competitive for fairly small jobs, even at considerable distances from the casting yard. The use of lightweight concrete extends the economical hauling range from casting yard to job site. The costs of the beams and slabs even on these small jobs show that, where properly used, prestressing makes economic sense.

JOB DESCRIPTIONS

One project was a two-story warehouse and office with 40-ft clear spans between two-story tilted up concrete walls (Fig. 1). Prestressed beams designed for 100 psf live load supported the second floor. Beams were 32 in. deep with a 20-in. top flange. The roof was carried on 26 in. deep beams with an 18-in. flange. The other project was a television studio and transmitting station. Here the beams were designed for a roof load including an allowance for ice fall-off from the tower. Sections of the beams and their significant design constants, together with the safety factors used in the design, are shown in Fig. 2.

In both jobs the beams supported precast assembled concrete plank floors. On the second floor of the warehouse, these plank received a 2-in. concrete topping and negative steel over the supports; on the roofs of both buildings the slabs had continuity steel in top grooves between slabs over the supports. These rods were bonded by a slurry of grout used to fill the key joints, but no topping slab was used, roofing being applied directly to the slabs.

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†Member American Concrete Institute, Partner, James & Honey, Consulting Engineers, Portland, Ore.

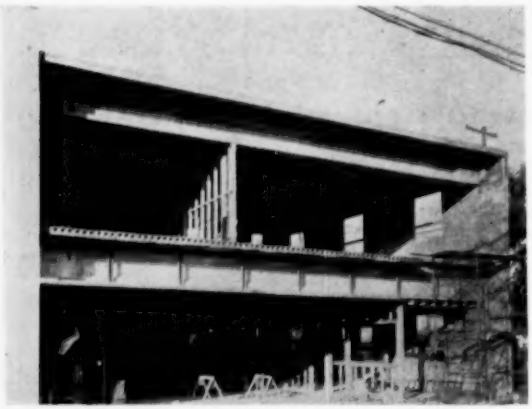


Fig. 1—Two-story warehouse with tilt-up walls utilized both precast prestressed beams and prefabricated floor and roof slabs

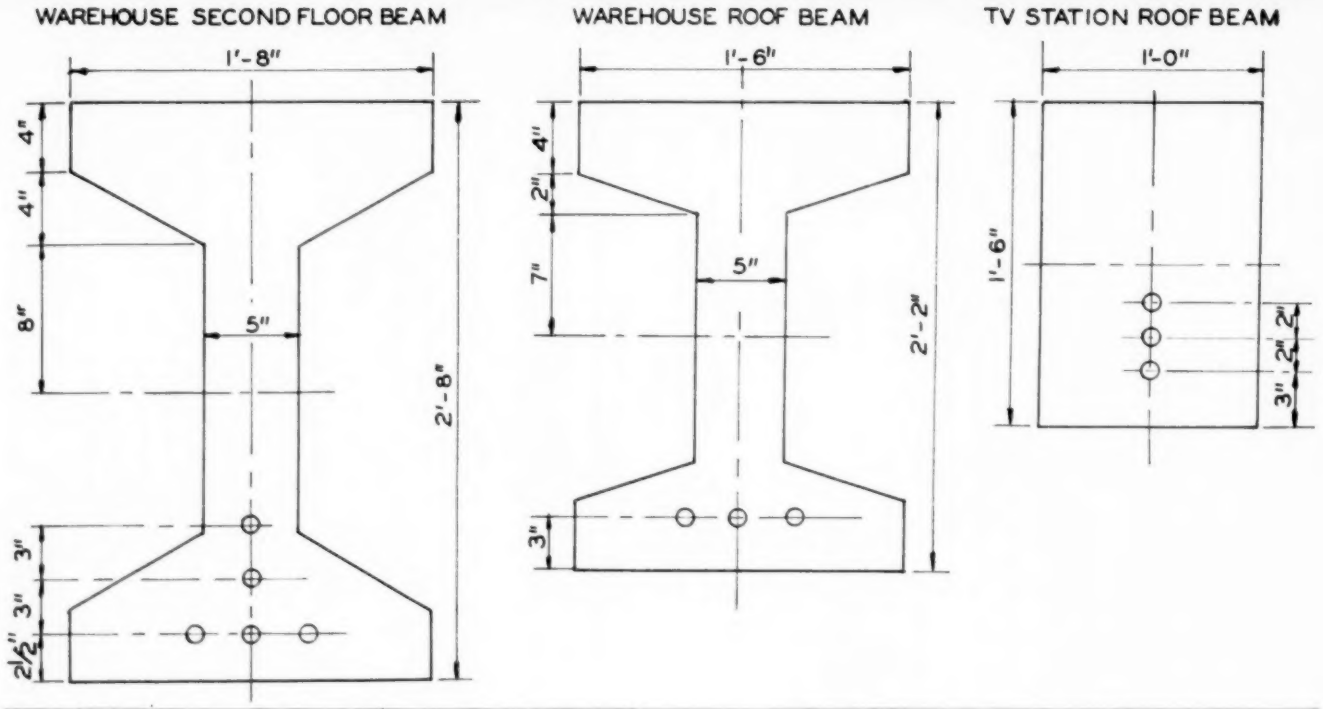
DESIGN OF BEAMS AND SLABS

There was nothing unusual about the slabs. They were made of lightweight expanded shale block with grouted-in reinforcing and a certain amount of prestress by Empire Prestress Concrete of Oregon, which has been manufacturing them for several years. They do have one advantage over some other types in that they can be locked together with a minimum of grouting due to a key joint on the edges at the top of the slabs. Also, continuity steel can be placed in these joints and continuity of a limited sort obtained without using any topping slab. They make a much more rigid and satisfactory diaphragm than precast elements which are not joined on the edges and not able to transmit moment between panels. This is important in seismic areas (such as the Pacific Slope) where any floor or roof which cannot function as a diaphragm for transmission of lateral forces is a decided liability in building design.

The beams for both jobs were designed with the same basic stresses:

$f_c' = 5000$ psi at 28 days (ultimate)
$f_c = 0.4\, f_c' = 2000$ psi (working)
$f'_{ci} = 2/3\, f_c' = 3300$ psi at transfer (strength required at time of stressing)
$f_{ci} = 0.6\, f'_{ci} = 2000$ psi (maximum stress at prestressing)
Final tension under load = 0
Tension at transfer $f_{ti} = -167$ psi (allowed in top flange at prestressing but mild steel provided to distribute)
Steel at transfer = $0.67 \times 250{,}000 = 167{,}000$ psi
Steel after losses = $167{,}000 \times 0.8 = 134{,}000$ psi

None of these basic design stresses bears comment except perhaps the allowance for losses in the steel. The concrete was expanded shale aggregate concrete weighing around 110 lb per cu ft. Since this type of concrete has a lower modulus of elasticity than gravel concrete of comparable strength and a proportionally higher deformation under load, it was decided to increase the allowance for creep and plastic flow from 15 percent of steel stress to 20 percent. Shrinkage was almost completely eliminated by steam curing



Beam	A_c, sq in.	I, in.^{4}	Z, cu. in	Initial prestress, lb	Final prestress, lb	Design moments, ft-lb						Cracking safety factor	Ultimate safety factor
						DL	Beam weight	LL	Total	Cracking	Ultimate		
Warehouse: Second floor	340	44,228	2770	317,000	254,000	97,400	47,200	263,000	407,600	579,600	985,000	DL + 1.65LL	DL + 3.2LL
Roof	260	21,880	1690	192,000	154,000	54,200	37,100	81,500	172,800	308,800	560,000	DL + 2.67LL	DL + 5.75LL
TV station: Roof	216	5830	648	181,000	145,000	29,400	14,700	41,200	85,300	122,500	257,000	DL + 1.9LL	DL + 5.16LL

Fig. 2—Beam sections, design constants, design moments, and safety factors

before tensioning and use of practically no-slump concrete, so it was not considered an important factor in contributing to prestress loss.

Shape, depth, width of flanges, and location of prestressing steel at the center of the beams are shown for the three beams in Fig. 2. It is beyond the scope of this article to discuss the theoretical considerations which led to the selection of the three different shapes. For a comprehensive discussion of shape selection the reader is referred to Guyon* and Magnel.†

However, there were some factors which affected the choice of these shapes. In the first place, the two jobs were 120 miles from the casting yards. The television station was on top of a small mountain with the access road so steep that trucks carrying materials had to be pulled part way by tractor. Erection on both jobs was handled by contractors unfamiliar with prestressed concrete construction. It was decided to make the beams in rugged shapes to stand the haul and erection. In addition, a certain percent of unstressed reinforcement was used to take care of load overhang in hauling and as insurance against rough handling. The author feels that a minimum amount of mild steel reinforcement in both top and bottom flanges is a good investment. Four # 5 bars or 1.20 sq in. were used in each flange of the I-beams and 0.93 sq in. in the rectangular beams.

There are, of course, many shapes and sizes of beams which will do a given job in prestressed design, but they fall mainly into three groups: I-beams, T-beams and inverted T-beams, and rectangular beams. Generalities are often dangerous and subject to many exceptions, but here are a few which we use as guides: I-beams are generally most efficient; they are also the most expensive to form and cast. T-beams are next in efficiency and only have one flange to form; rectangular beams are lowest in efficiency, highest in dead weight, but they are the cheapest to form and easiest to cast. The number of beams required of a given shape and size is a factor. A fancy I-shape is more reasonable where the form can be re-used enough times to cut the unit forming cost down. In the case of the TV station only four beams were needed, so it was not worth while designing anything hard to form; it was cheaper to use extra concrete and steel. However, on longer beams with heavier loads the rectangular shape is too wasteful to be considered.

Basic safety factors used in our design were:

Cracking moment = dead load moment plus 1.5 times live load moment

Ultimate moment = dead load moment plus 3.0 times live load moment

The cracking moment is computed based on allowing 700 psi in tension with no allowance for the mild steel. The ultimate moment is simply $A_s f_s jd$. An examination of the safety factors listed for each beam in Fig. 2 will show that these are not the exact factors arrived at in each case; beams were designed and then checked to see that minimum safety factors required by standards were met. In addition the shear and maximum principle tensile stress, and the bond stress were checked. These three stresses were so small

*Guyon, Y., *Prestressed Concrete*, Chapter 9, John Wiley and Sons, Inc., New York, N. Y.
†Magnel, Gustave, *Prestressed Concrete*, 2nd Edition, Chapter 3, Concrete Publications, Ltd., London, England.

as to be negligible and not a significant factor in these designs. A series of deflection calculations were also made (see properties of prestressed beams, Fig. 2) and were a valuable check on the prestressing forces and the assumed E_c values:

Warehouse second floor beam:

Computed camber due to prestress $\Delta_1 = \dfrac{5(PEL^2)}{48\,EI}$ $= \dfrac{5(317{,}000)(11.7)(480)^2}{48(2{,}000{,}000)(44{,}228)} = 1.01$ in.

Deflection due to beam dead load $\Delta_2 = \dfrac{5(ML^2)}{48\,EI}$ $= \dfrac{5(47{,}200)(12)(480)^2}{48(2{,}000{,}000)(44{,}228)} = -0.15$ in.

Net camber after prestressing $= 0.86$ in.

This compares with the average measured value of camber for four beams of 0.81 in.

Warehouse roof beams:

Computed camber due to prestress $= \dfrac{5(192{,}000)(10)(486)^2}{48(2{,}000{,}000)(21{,}880)} = 1.08$ in.

Deflection due to beam dead load $= \dfrac{5(37{,}100)(12)(486)^2}{48(2{,}000{,}000)(21{,}880)} = -0.25$ in.

Net camber after prestressing $= 0.83$ in.

This compares with the average measured value of camber for six beams of 0.74 in.

Television station roof beam:

Computed camber due to prestress $= \dfrac{5(181{,}000)(4)(336)^2}{48(2{,}000{,}000)(5830)} = 0.73$ in.

Deflection due to beam dead load $= \dfrac{5(14{,}700)(12)(336)^2}{48(2{,}000{,}000)(5830)} = 0.18$ in.

Net camber after prestressing $= 0.55$ in.

This compares with average measured value of camber of four beams of 0.50 in.

Modulus of elasticity of expanded shale aggregate concrete

The value of 2,000,000 psi used in the deflection and camber calculations is thus seen to be justified for the instantaneous modulus of this concrete. This confirms the values obtained in research sponsored by Empire Building Material Co. on the same aggregate at the Oregon State College Engineering Experiment Station and published in August, 1951, in *Bulletin* No. 30, "Expanded Shale Aggregate in Structural Concrete." Using aggregate from the same manufacturer they arrived at a formula for the modulus of elasticity of: $750{,}000 + (250 \times f_c')$, which for 5000-psi concrete gives E equal to 2,000,000 psi. This value is substantially different from values reported by J. A. Murlin* and Fred E. Koebel.† The author believes that E values are very much a matter of the particular aggregate and mix used, and should be determined for each particular lightweight aggregate used. Once E is determined it is used in another part of the operation, namely the prestressing. In computing the amount of elongation of the wires to get the desired prestress there is a correction made for nonsimultaneous prestressing. That is the loss in prestress in all units previously stressed as each new one is

*Murlin, J. A., "Lightweight Concrete for Lower Construction Costs," ACI JOURNAL, Sept. 1951, *Proc.* V. 48, pp. 37-44.

†Koebel, Fred E., "Lightweight Prestressed Concrete," ACI JOURNAL, Mar. 1954, *Proc.* V. 50, pp. 585-596.

stressed. To correct for this it is common practice to overstress all cables an amount calculated to compensate for this loss. This amount of overstress is based on the ratio of E (steel)/E (concrete), and for this aggregate using E_c as 2,000,000 the overstress is double that for gravel concrete. In the cases of the three beams shown it only amounted to increasing the overstress allowance on the average from 1.5 to 3.0 percent of final total prestressing force, so does not affect the operation significantly.

BEAM MANUFACTURE

Forms

Forms were $\frac{3}{4}$-in. plywood sides and bottom. One set of sides sufficed for each of the three beam sections, although the forms for the warehouse roof beam, which required six castings, required some repairs. A note on sections—it is wise to avoid too flat a slope in the bottom flange since this makes it hard to fill and causes rock pockets. It was found wise to chamfer all corners, since stripping green concrete to set-up for another casting is hard on right angle corners. Also, at the time of prestressing when the beam cambers, it throws all the beam weight on the ends at the bottom; a green sharp corner tends to chip off.

Casting

Concrete was placed from a Scoopmobile; being practically no-slump concrete it was vibrated hard to push it down through the 5-in. webs. There was no form trouble except when a workman forgot one of the bolts in the bottom flange; a bulge formed and had to be ground off later. Forms must be built and braced exceptionally well to take this kind of use. Care must be taken not to move any of the prestressing cables during casting and for this reason form vibrators would have been more desirable than the regular vibrators used. The position of the cables *must* be maintained and stirrups and mild steel should be tied in to them so they cannot move. Another reason for external vibration is to avoid damaging the sheathing around the cables causing them to bind or freeze in the sheath.

Jacking and grouting

All cables were composed of twelve 0.196-in. diameter wires and were stressed from one end using a Freyssinet jack. They were anchored with standard Freyssinet cones, and grouted through the center hole after all cables in each beam were stressed. Before grouting the camber was verified with the results previously given. This check on predicted versus measured camber is a valuable one in the author's opinion since it verifies the accuracy of the prestressing forces and their location and effect on the beam.

One thing to bear in mind is to leave enough room on the jacking end to maneuver the jack into position. Generally speaking it isn't economical to provide enough bottom steel to move the beams before stressing, so clearance must be provided at one end for the jack. Another thing noticed was that pressure gages tend to get out of adjustment in shipping or use and elongation

should be checked both on the live end and slip on the dead end to assure the correct net elongation desired.

BEAM TEST

One of the TV station roof beams was stressed before concrete had attained a strength of over 3300 psi. The morning after stressing it was observed that the lower anchorage cone on the jacking end had slipped in ½ in. and caused a small piece of the end of the beam to shear off. There was a concentration of stress in this region due to the two 3-in. O. D. pipe inserts in the bottom of the beam 1½ in. high and 4 in. from the end of the beam. It was felt that these inserts aggravated the stress concentration back of the lower anchorage cone and it was more than the green concrete could take. In the other three beams where the concrete had higher strength before stressing there was no distress in this region.

It was decided to patch the end of the beam and load test the beam to dead load plus 1.5 times live load in the casting yard. The patch was made using a proprietary coating on the joint to aid bond, and after it had become tacky troweling on the high strength patching material. Then the patch and the beam were cured in the steam room for 48 hr. The beam was loaded with pallets of concrete block. Deflection was measured by reading an engineers scale through a Dumpy level; using the 50-scale, readings were obtained to 0.02 in. The ends of the beam were checked for settlement and 0.06 in. detected under load at one end. Readings were adjusted for half of this amount at the center. Fig. 3 shows location of the loads and a load-deflection curve for the beam. The patched end and the whole beam were carefully examined for cracks or signs of distress during and after the test and none was detected. After removal of load, recovery was 100 percent and the beam was shipped to the job.

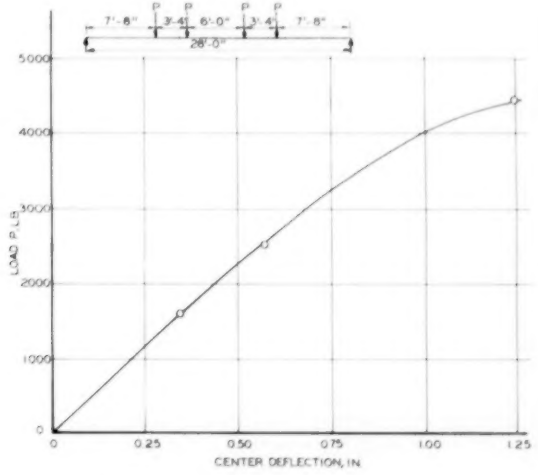


Fig. 3—Load-deflection curve for 28-ft roof beam

Discussion of the effects of cable slippage on the cracking and ultimate safety

Factors—A brief analysis of the effects on the strength and safety factors of this beam is hereafter presented.

Prestress force after ½-in. slippage of one cable (2.14 in. is net extension to get 167,000 psi):

Stress in one cable is 0.362 sq in. × 167,000		=	60,500 lb
Net loss in No. 3 cable is (0.5/2.14) × 60,500		=	− 14,100 lb
Stress retained in No. 3 cable		=	46,400 lb
			181,000 lb
			− 14,000 lb
New total prestress force		=	167,000 lb
New eccentricity:	46,400 × 6	=	278,000
	60,500 × 4	=	242,000
	60,500 × 2	=	121,000
			641,000 = 3.84 in.
			167,400

New cracking moment—

Assuming 20 percent prestress loss then 167,000 × 0.80 = 134,000

Allowing 700 psi in concrete tension at cracking:

$P/A + Pc/S + M/S = -700$

$134{,}000/216 + (134{,}000 \times 3.84)/648 - M/648 = -700$

Solving for M = 114,000 ft-lb (cracking moment)
− 44,100 ft-lb (dead load moment)

$$\frac{69{,}900 \text{ ft-lb}}{44{,}1000 \text{ ft-lb}} = 1.7 \times \text{live load moment}$$

Thus the cracking safety factor is seen to change from DL plus 1.9 × LL to DL plus 1.7 × LL which is still greater than the design standard of DL plus 1.5 × LL. The ultimate safety factor, being based on the area of steel × steel ultimate strength × j × effective steel depth, changed not at all but remained DL plus 5.16 × LL. This characteristic of beams with bonded prestressing is reassuring, and worthy of some consideration.

HAULING AND ERECTION OF BEAMS AND SLABS

Fig. 4 shows how the beams were hauled. Note that the permissible overhang on the ends is limited by the cantilever moment stress in the top which is *additive* to the effect of cable eccentricity. In addition it was felt that on a long haul like this the possibility of impact and vibration from whipping of the cantilever end could produce more moment so an allowance of 100 percent was made for this effect. In one beam it was found advisable to add additional mild steel in the top near the ends to carry this hauling overhang moment. There was no damage to beams during hauling. Erection was by single crane using a spreader. This spreader, as can be seen in Fig. 4, was too short to do much good and there were bending stresses in the lifting hooks, which in this case were 1-in. diameter reinforcing bars. Since the casting yard cannot control the contractor's lifting equipment, it was decided to place the lifting hooks on a slope on future jobs to correspond to the angle

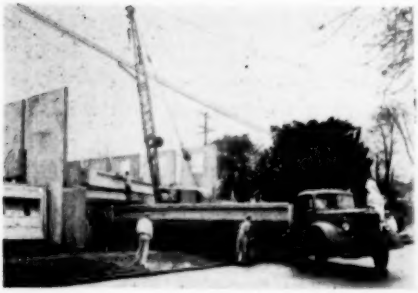


Fig. 4—Erection of beams on two-story warehouse

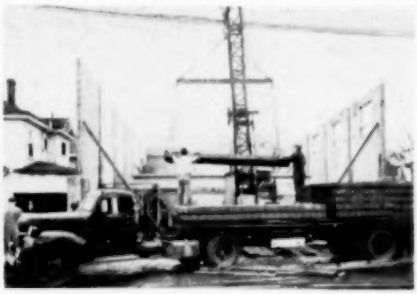


Fig. 5—Erection of block slabs on two-story warehouse

of lift lines and advise the contractor to arrange his rigging to match this slope. This has worked out well on later jobs.

Method of hauling the slabs is shown in Fig. 5. This worked out well, but there was a higher percentage of breakage on this particular trailer due, it was felt, to its extreme flexibility in the bed. Canvas slings were later used to handle the slabs (Fig. 6) and resulted in less edge chipping and faster, easier handling. Mention should be made that in computing the maximum truck loads of both beams and slabs the reduction in weight of almost 30 percent due to use of expanded shale aggregate was very important. Maximum wheel and axle loads are strictly regulated in Oregon and loadings were based on these regulations, each load being weighed before departure. Fully ⅓ more truck round trips of 240 miles would have been required with heavy concrete.

SUMMARY OF COSTS

Price of the block slabs at the plant is standardized at 65 cents per sq ft for the 4-in. depth and 85 cents per sq ft for the 6-in. depth. Five cents per sq ft is allowed for city delivery. Erection costs vary depending on the area and the height to which slabs must be raised, as well as the efficiency of the contractor in placing the slabs. A fair range would be 5 to 10 cents per sq ft. These prices will cover the grouting of side key joints between slabs

Fig. 6—Setting 6 in. deep block slabs with canvas slings and spreader on television station roof

and the ends over bearings. For a 2-in. topping slab there is an added 20 cents per sq ft.

Beam prices are harder to tie down without going into considerable detail. Cost of each job is broken down into amounts of prestressing steel, concrete, and mild steel reinforcement; forming cost based on shape and size of member and number of form re-uses; labor in placing, stressing, and grouting the prestressing steel; costs for engineering and shop details; handling and loading costs; selling, overhead, and profit. All these will vary for different casting yards and different localities. From a series of recent jobs furnished by Empire Prestress Concrete of Oregon, for whom the writer's firm is consulting engineers, the following table of general prices apply.

Span range in ft	Cost in dollars per sq ft of tributary area
25–30	0.50
30–40	0.65
40–50	0.80

These are based on roof loads and lightweight concrete construction, with a total of about 50 lb per sq ft dead plus live load, and a minimum of four form re-uses. For example, consider a beam spanning 50 ft clear with center to center spacing of 16 ft. Beam cost would be estimated at $0.80 \times 16 =$ $12.80 per lineal ft of beam. The table stops at 50 ft span length because this is about the limit for casting yard fabrication.

CONCLUSIONS

1. Greater clear spans at lower cost are possible through lightweight precast and prestressed concrete beams and slabs fabricated at casting yards.
2. Lightweight concrete extends the economical hauling range from casting yards.
3. Engineering and close control are practical on both small and large jobs manufactured at yards.
4. These products and methods make concrete roof and floor structures more competitive with other materials.

Strength of Continuous Prestressed Concrete Beams Under Static and Repeated Loads*

By T. Y. LIN†

SYNOPSIS

Basic principles underlying the behavior of statically indeterminate prestressed concrete structures were investigated by testing four continuous beams, each 50 ft long. The cracking and ultimate strengths of these beams were measured and evaluated for static loads as well as for repeated loads up to 5,000,000 cycles. Beam reactions, deflections, and strains were recorded for analysis. The effects of adding nonprestressed mild steel reinforcement were studied.

INTRODUCTION

Questions have often been raised regarding the behavior of statically indeterminate prestressed concrete structures, as distinguished from statically determinate structures on the one hand and from indeterminate reinforced ones on the other. The condition of statical indeterminacy in prestressed concrete imposes several new problems, foremost among which is the accuracy and applicability of the elastic theory such as used for analysis of other statically indeterminate structures.

All engineers agree that, under working loads, prestressed structures follow the elastic theory more closely than reinforced concrete, owing to the nonexistence of cracks. But there have been questions as to whether the cracking of concrete can be predicted by the elastic theory, using the flexural tensile strength as determined by plain concrete specimens.‡ Some engineers believe that previous to cracking there will be sufficient plastic behavior of concrete in tension so as to delay the appearance of cracks.§ Hence one purpose of these tests was to determine the accuracy of the elastic theory for computations in continuous prestressed beams right up to cracking.

Regarding the ultimate strength of statically indeterminate prestressed structures, there is general agreement on the applicability of the plastic hinge theory** as it has been applied to other materials. However, the accu-

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†Member American Concrete Institute, Associate Professor of Civil Engineering and Associate Research Engineer, Institute of Transportation and Traffic Engineering, University of California, Berkeley, Calif.

‡Rogers, G. L., "Validity of Certain Assumptions in the Mechanics of Prestressed Concrete," ACI JOURNAL, Dec. 1953, *Proc.* V. 50, pp. 317-332.

§Freyssinet, E., "Importance et Difficultés de la Mécanique des Bétons," *Proceedings*, First International Congress on Prestressed Concrete, Ghent, Belgium, 1951. (English translation in *Magazine of Concrete Research*, No. 8, 1951, Cement and Concrete Assn., London)

**Guyon, Y., "Statically Indeterminate Structures in the Elastic and Plastic States," *General Report*, First International Congress, International Federation of Prestressing, London, 1953.

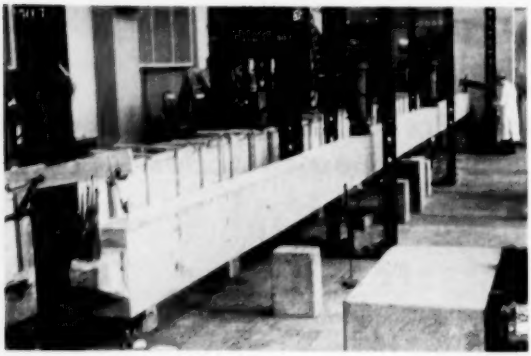


Fig. 1—Test set-up for beams

racy of that theory is questioned when the members fail in compression in the concrete. Furthermore, under the action of repeated loads, the mode of failure may be entirely different, and the plastic theory may not be applicable at all. Therefore, these tests were designed to determine the ultimate strength under repeated as well as static loads.

In addition to the usual measurement of strains and deflections, attempts were made to determine the actual moments existing in the beams. This was accomplished by measuring the reactions to the beams, which gave a direct computation for the moments and hence a reliable assessment of the validity of the various theories.

It is known that by properly placing nonprestressed reinforcement in a prestressed concrete beam, the strength of the beam can be profitably increased. Although the effect of such bars on the start of initial hair cracks is regarded as minor, they help to limit the extension and concentration of cracks and to increase the ultimate strength under both static and repeated loads. Their effects in a statically indeterminate structure were evaluated in these tests.

The entire project was carried out in the Concrete Laboratory of Prof. G. Magnel of the University of Ghent, Belgium, with the author in charge.

DESCRIPTION OF BEAMS

Four 50 ft long, continuous prestressed beams were made in the laboratory (Fig. 1). All four beams, A, B, C, and D, were identical with the exception that two beams, B and D, contained nonprestressed mild steel bars while

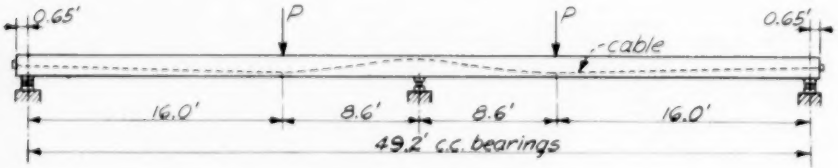


Fig. 2—Beam elevation

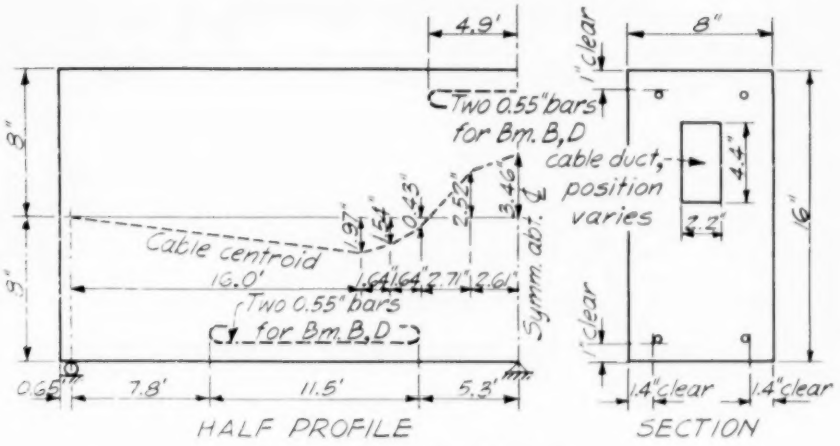


Fig. 3—Beam profile and section

A and C did not. Beams A and B were tested for static loads only; beams C and D were subjected to repeated loadings and measured for static loads between intervals.

An elevation of the beam is shown in Fig. 2. The beam had a uniform cross section of 8 x 16 in. (Fig. 3), and was prestressed with a Magnel cable with varying eccentricity along the length of the beam. The cable consisted of 32 parallel wires of 0.196-in. diameter with an effective prestress of 120,000 psi. The cable profile was designed so that both at transfer of prestress and under the working load, little or no tensile stress would exist at any point along the beam. The working load consisted of two concentrated loads of 11,000 lb each applied as shown in Fig. 2. To simplify the problem, a concordant cable was chosen so that under prestress there would be no tendency for the beam to deflect away from its supports.

Five 0.47-in. diameter mild steel closed stirrups spaced at 3-in. centers and supported with four 0.55-in. diameter bars at the corners were provided to reinforce the ends of all beams. For beams B and D, two mild steel bars of 0.55-in. diameter were added over the center support and under the loads (Fig. 3). Steel bearing plates were provided at all points of loading and reactions; all plates were 6 in. long (except 8 in. long for the center support) and extended the full 8-in. width of the beam. A hinged support was furnished at the center, while rollers were employed for the end reactions.

Materials

The concrete for the beam was designed to possess a minimum strength of 6400 psi based on 8-in. cubes at 28 days. This strength is equivalent to about 5000 psi as determined by 6 x 12-in. cylinders. Eight cubes were made for each beam. They were cured in the laboratory under conditions similar to the beams. Two specimens were tested at 7, 14, 28 days, and at day of rupture of the beam. The average results of each two specimens are listed in Table 1.

TABLE 1—CUBE STRENGTH OF CONCRETE, PSI

Time	Beam A	Beam B	Beam C	Beam D	Average for four beams
At 7 days	5120	5680	5790	5500	5520
At 14 days	5900	6960	8200	7520	7150
At 28 days	6730	7680	8700	7800	7730
At rupture	8200	8420	8550	7950	8280

Three beam specimens of 6 x 6 x 24 in. were tested on a span of 20 in. to determine the flexural tensile strength at the day of cracking for each beam. As can be seen from Table 2, the individual values varied from 710 up to 1040 psi.

TABLE 2—FLEXURAL TENSILE STRENGTH OF CONCRETE, PSI

Specimen	Beam A	Beam B	Beam C	Beam D	Average for four beams
No. 1	830	830	800	710	
No. 2	1040	740	800	770	
No. 3	900	840	800	830	
Average	920	800	800	770	820

To determine the modulus of elasticity for the concrete, three specimens of 8 x 8 x 20 in. were cast for beam A and tested at various ages (Table 3). In addition, one specimen of 4 x 4 x 24 in. was cast for each beam and measured for initial modulus by a Bouche's soniscope; results are recorded in Table 4. A typical stress-strain curve for the concrete under direct compression is shown in Fig. 4.

TABLE 3—SECANT MODULUS OF CONCRETE, BEAM A, PSI
(By direct compression test)

Time	At 700 psi	At 1400 psi
At 14 days	5,300,000	4,540,000
At 28 days	5,450,000	4,510,000
At cracking	5,420,000	4,700,000

TABLE 4—MODULUS OF ELASTICITY OF CONCRETE, PSI
(By vibration)

Time	Beam A	Beam B	Beam C	Beam D
At 7 days	6,130,000	5,910,000	5,670,000	5,930,000
At 14 days	6,180,000	6,100,000	5,850,000	6,070,000
At 28 days	6,250,000	6,130,000	5,930,000	6,130,000

The cement was Type I portland cement made in Belgium. It was subjected to standard tests as prescribed in Belgium and found to meet all the requirements. Initial set of the cement occurred after 45 min while final set took place after 4½ hr. For compression tests, a water-cement ratio of 0.28 was used and 15 cubes of 2.8 in. were made of 1:3 mortar using normal standard sand. The cubes were mechanically compacted and cured in 68 F water

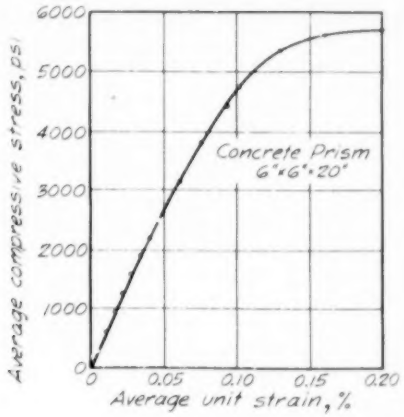


Fig. 4—Typical stress-strain diagram for concrete

until testing. For direct tension tests, 15 briquettes were made under the same conditions. The average strengths of the mortars are listed in Table 5.

TABLE 5—STRENGTH OF 1:3 MORTARS

	3 days	7 days	28 days
Compressive strength, psi	5750	6440	9160
Tensile strength, psi	384	390	492

One batch of aggregates, consisting of river sand and gravel, was used for all beams. Measurement was made by volume and the proportion by weight was computed to be:

Gravel	Sand	Cement	Water
3.14	1.74	1	0.38

This produced concrete with slump varying from ¼ to ½ in. The consistency of concrete was controlled and measured by the Vébé test, giving values between 10 and 13 sec. Unit weight of concrete averaged 155 lb per cu ft. Cement content was 7.2 sacks per cu yd.

Grout for the cableway was made with equal volumes of cement and sand with a water-cement ratio of 0.5. An air-entraining agent equal to 0.3 percent of the weight of cement was added to the grout to increase its fluidity. Strength of the grout as given by 4-in. cubes averaged 5000 psi at the day of rupture of the beams.

The cold-drawn prestressing wires were of Belgian make. Tested specimens showed remarkably uniform results, both in strength and strain characteristics. These 0.196-in. diameter wires possessed an ultimate strength of 256,000 psi, with a proportional limit at 200,000 psi, and a yield point of 228,000 psi measured by the 0.2 percent offset method. Total elongation was 5 percent in a gage length of 8 in. The modulus of elasticity was 29,000,000 psi up to the proportional limit. A typical stress-strain diagram is shown in Fig. 5.

The nonprestressed mild steel bars had an ultimate strength of 59,000 psi, with a yield point of 45,500 psi. On a gage length of 8 in. elongation was 16 percent. The modulus of elasticity was 28,400,000 psi. A stress-strain diagram for the bars is shown in Fig. 6.

Manufacture

All beams were cast in place to avoid the difficulty of handling continuous members. After the forms were erected for each beam, two retractable rectangular rubber tubes, one on top of the other, were placed in the forms and positioned by ¼-in. temporary steel bars inserted

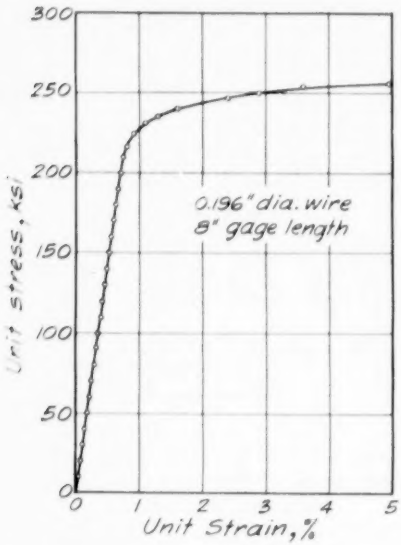


Fig. 5—Typical stress-strain diagram for wires

through the side forms. The circular hole in the center of each rubber core was stiffened with steel rods which were withdrawn soon after concreting.

Immediately after final set of the concrete, the rubber tubes were pulled out, leaving a continuous duct of 2.2 x 4.4 in. in the concrete. The beams were sprayed with water for the first 24 hr and the side forms stripped after at least 3 days.

The Magnel cable was prepared with the 32 wires spaced by 1.8 x 3.6-in. steel separators, dividing the wires into eight rows of four. One separator was placed at each end and at each point of bending of the cable—a total of 11 separators per beam. The wires were grouped together at the leading end and inserted into a steel nose to be pulled through the duct in the beam. Before inserting the cable the duct was first cleared of any debris.

Prestress was applied to each beam when it was 14 days old. Because of the reverse curvature of the cable, the wires were tensioned simultaneously from both ends to minimize the frictional loss. The amount of frictional loss was estimated* by taking 0.3 as the co-

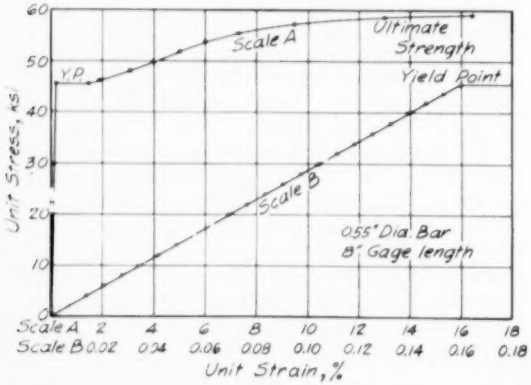


Fig. 6—Typical stress-strain diagram for mild steel bars

*Cooley, F. H., "Friction in Post-Tensioned Prestressing Systems," *Research Report*, Cement and Concrete Assn., London, 1953.

efficient of friction between the wires and concrete and by assuming the wobble effect of the duct as 0.0005 radian per ft. This gives a maximum loss at the center support of 6.3 percent, 4.8 percent of which was due to curvature and 1.5 percent due to wobble effect.

Loss of prestress due to slipping of the anchorage wedges was estimated as follows. Assuming a slipping of 0.1-in. for each end, the unit shortening in a 52.5 ft length of wire is $(2 \times 0.1)/(52.5 \times 12) = 0.000318$, which, for a modulus of 29,000,000 psi, indicates a loss of $0.000318 \times 29{,}000{,}000 = 9200$ psi, or $9200/142{,}000 = 6.4$ percent.

To overcome the frictional loss, the wires were stressed at the jacks to 6 percent above the desired initial value of 142,000 psi so that 100 percent stress would be obtained over the center support. When anchoring the wires, the loss of 6 percent due to anchorage slip would take place, and then the stress at the ends of the wires would also be 100 percent. The wires were tensioned in pairs by a Magnel jack at each end. The amount of prestress was measured by a dynamometer or a pressure gage and checked by the elongation of the wires. The average amount of total initial elongation was 3.46 in., while the sum of the slippages for two wedges averaged 0.17 in. Thus a net elongation of 3.29 in. was obtained in a length of 52.5 ft, which indicated an average stress of $3.29/(52.5 \times 12) \times 29{,}000{,}000 = 151{,}000$ psi.

The amount of anchorage slip actually differed for the beams:

	Beam A	Beam B	Beam C	Beam D
Average wedge slippage (total of two ends), in.	0.21	0.21	0.16	0.11

The difference was attributed to the anchorages for beams C and D, which were salvaged from beams A and B. It was possible that the wedges, being used for the second time, slipped less than new ones. Corresponding to the average slip for each beam, the initial prestress was 1.6 percent higher for beam C and 3.3 percent higher for beam D, as compared to beams A or B.

Loss of prestress due to creep and shrinkage in concrete and creep in steel was not measured because from previous experience in the laboratory, for the same kind of steel and concrete, a loss of 15 percent was known to be a close estimate.

After prestressing, the cable duct was grouted by injecting mortar through the holes provided at the ends of the beam, using a pressure head of 7 ft. After the beams were broken, examination of the grout showed solid encasement of all wires.

INSTRUMENTATION

Static loads were applied to each beam by two 30-ton hydraulic jacks connected to an Amsler hydraulic testing machine. The jacks acted against steel frames anchored to the laboratory floor through large screws in the heavy prestressed concrete floor slab.* For the pulsating loads, one Amsler hydraulic pulsator was employed. The pulsator has a capacity to pump 9 cu in. of oil per cycle under a pressure of 5700 psi. The maximum and minimum loads exerted by the pulsator were indicated by manometers. The pulsating frequency was 250 cycles per min.

For measurement of static reactions, Losenhausenwerk compression meters of the manometer type were installed at each end of beams A and B. These meters were first calibrated with a testing machine and their accuracy was

*Riessauw, F. G., "The New Equipment of the Laboratory for Reinforced Concrete at the University of Ghent for Static and Dynamic Tests on Construction Members of Large Size," *Precontrainte* (Brussels), No. 1, 1952.

found to be within 20 lb. When compressed, these meters shortened a maximum of 0.15 in., which settlement was within permissible limits for the beams. For beams C and D, subjected to pulsating loads, Philips compression meters of the dynamometer type were employed. These were also calibrated before and after the tests, and an accuracy of 50 lb was indicated. Their shortening under load was practically zero.

Strains in the concrete were measured by electric resistance gages. First, the surface of the concrete receiving the gages was ground smooth and cleaned with acetone. Then one coat of Philips strain gage glue was spread on it. After a few hours, the gages were attached with the same glue. For beams A and B, Baldwin SR-4 strain gages of the A1 type were used while C1-type gages were used for beam C. For beam D, similar Huggenberger gages of the BP2 type were used. The main purpose of these gages was to detect the appearance of cracks. Hence, they were placed on 4- and 8-in. intervals on the tension side of the beam around the sections of maximum positive and negative moments (Fig. 7). Baldwin strain gage indicators were employed for reading strains in beams A and B, Philips indicators for beams C and D. Philips oscillograph units were used for reading the strains, reactions, and deflections during the pulsating loadings. For beams A and B, Huggenberger tensometers were installed between the strain gage points, but they were found to be unnecessary and were omitted for beams C and D.

Deflection meters reading to 0.0003 in. were placed at eight points along the beam, both during prestressing and during static loadings. For the pulsating loads, a special device consisting of two such deflection meters was installed at one midspan point to measure the maximum and minimum deflections of the beam under pulsating loads.

TEST PROGRAM

Beam A was tested only for static loads. First, the working load ($P = 11{,}000$ lb each) was applied in five equal increments while readings were taken of all meters and gages. The test was repeated twice. Then the beam was loaded until several cracks were seen over the center support (up to $P = 16{,}500$ lb each) in increments of 1100 and 550 lb. Then it was loaded until cracks

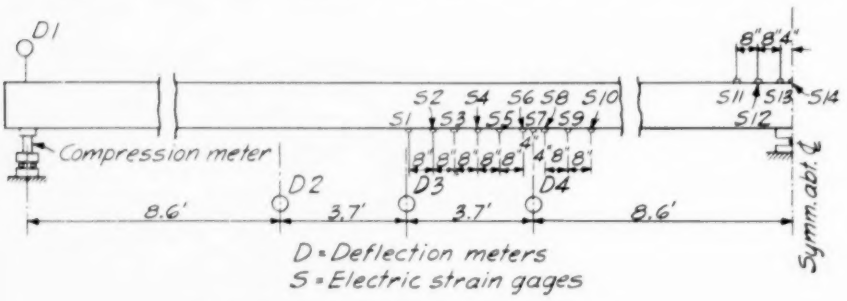


Fig. 7—Typical gaging layout for one span of beam

occurred under the loads (up to $P = 26{,}500$ lb each), then loaded up to rupture, all in increments of 550 lb. The commencement of cracks was detected by the erratic behavior of the strain gages and searched for with magnifying glasses. After cracking became general, strain gage readings were abandoned. All reaction and some deflection readings were taken up to rupture.

Beam B was subjected to exactly the same sequence of tests as beam A to confirm the results obtained for beam A, and to determine the effect of adding nonprestressed reinforcement.

Beam C was subjected to repeated loads to determine the effect of fatigue. The pulsation started with a range of 0.5 to 1.0 times working load ($P = 5500$ to 11,000 lb) as indicated by the manometers, then a range of 0.5 to 1.2 working load, then 0.5 to 1.4 working load, etc. Each time the upper limit was increased by 0.2 working load (increments of $P = 2200$ lb each), while the lower limit was raised only when the full capacity of the pulsator was reached. For each range of pulsation about 500,000 cycles were applied. The record of pulsation is shown graphically in Fig. 8.

During the pulsation tests, oscillograph and deflection readings were taken to determine the dynamic increment of the pulsating loads. The oscillograph readings were not consistent, but the maximum and minimum deflections recorded by the deflection meters were found to be very accurate. They indicated that due to dynamic effect, the actual minimum load on the beam averaged 7 percent lower while the actual maximum load averaged 6 percent higher than given by the manometers.

A static test was run before starting the pulsation. Then before every increase in the pulsating loads, a static test was run to record the behavior

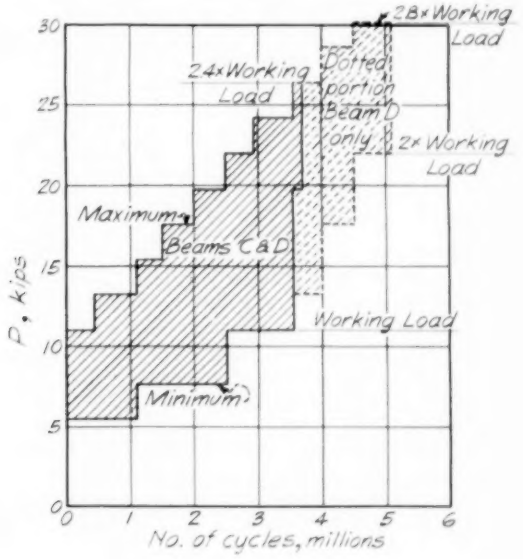


Fig. 8—Record of pulsation tests, beams C and D

of the beam. After the beam showed evidence of failure due to the breakage of a number of wires, it was tested statically up to total rupture.

Beam D was tested similarly to beam C, again for confirmation of some results and for determining the effect of adding nonprestressed reinforcement.

CRACKING AND ULTIMATE LOADS—MEASURED VERSUS COMPUTED VALUES

Table 6 compares the measured and computed values of the cracking and ultimate loads of the beams. The cracking loads were those producing the first observed hair cracks. As expected, cracking occurred first over the center support and later under one of the loaded points. This was the same for all four beams.

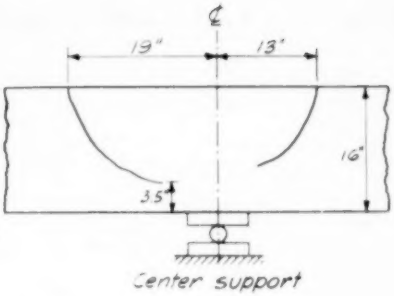
Rupture of beams A and B resulted from the crushing of concrete over the center support. Prior to rupture, the vertical tensile cracks over the support developed into diagonal cracks, reducing the depth of concrete under compression (Fig. 9).

Failure of beams C and D under repeated loads was first evidenced by the breaking of wires over the center support as a result of fatigue. After about half of the wires were broken over the center support (no wires broke elsewhere), the beams were subjected to static tests until they collapsed in the compression of concrete under one of the loads. This was explained by the fact that there was not enough force to crush the concrete over the center support where half of the wires were already broken. Final examination of the beams showed that 16 wires had broken in beam A and 17 in beam B. All exhibited crystalline fatigue failures and all were broken exactly at the separator over the center support.

TABLE 6—CRACKING AND ULTIMATE LOADS—MEASURED VERSUS COMPUTED VALUES

Beam No.	A	B	C	D
Type of loading	Static	Static	Repeated	Repeated
Mild steel bars	Without	With	Without	With
Design load, kips	11	11	11	11
Cracking load, kips				
Center support:				
Measured	14.8	15.8	15.8	17.6
Computed (elastic theory)	15.5	15.5	14.4	14.4
Midspan:				
Measured	24.2	23.1	22.4	23.1
Computed (elastic theory)	24.9	24.9	23.1	23.1
Ultimate load, kips				
Static loading only:				
Measured	39.2	45.8		
Computed, plastic hinge theory	41.5	45.8		
Repeated loadings:				
First broken wire			24.2	28.6
No. of cycles, million			3.2	4.4
About half wires broken			26.4	30.8
No. of cycles, million			3.7	5.1
Static test to destruction			33.0	40.2

Fig. 9—Main cracks before rupture, beam A



COMPUTATION FOR CRACKING UNDER STATIC LOADS

The concordancy of the cable can be checked by taking statical moment of the M/EI diagram due to prestress. It can be easily shown that there is no external reaction due to prestressing and the cable is concordant. Thus the stresses in the concrete due to prestress can be simply computed from the formula

$$f_c = \frac{F}{A} \pm \frac{Fec}{I}$$

where F = prestress = $f_s A_s$ = 120,000 psi × 0.963 sq in. = 115,500 lb
A = concrete area = 8 × 16 = 128 sq in.
e = eccentricity of cable = 1.97 in. under the load and 3.46 in. over the center support
c = 8 in.

$$I = \frac{8 \times 16^3}{12} = 2730 \text{ in.}^4$$

To be strictly correct, A, E, I, and c should refer to the net concrete section before grouting; after grouting, they should refer to the transformed section. But, for simplicity in presentation, the gross section of concrete will be used at all times, although errors amounting to a few percent are involved in this approximation.

All extreme fiber stresses for the maximum $+M$ and $-M$ sections are computed and listed in Table 7.

Due to the weight of the beam, at 140 lb per ft, the moment can be computed by the usual elastic theory for beams, giving

$$-M \text{ over center support} = \frac{wL^2}{8} = \frac{140 \times 24.6^2}{8} = 10.5 \text{ kip-ft}$$

$$+M \text{ under the load} = 2.7 \text{ kip-ft}$$

The stresses are again computed by Mc/I.

For two loads, one P at each span, the moment as given by the elastic theory is

Maximum $-M = 0.188\,PL = 0.188 \times 24.6\,P = 4.63\,P$
Maximum $+M = 0.106\,PL = 0.106 \times 24.6\,P = 2.61\,P$

With $P = 11{,}000$ lb, the resulting stresses due to prestress and dead and live loads are listed in Table 7.

TABLE 7—COMPUTATION FOR STRESSES IN CONCRETE, PSI*

Point	Under load		Over center support	
Fiber	Top	Bottom	Top	Bottom
Prestress	− 235	−1565	−2070	+ 270
Beam own weight	− 95	+ 95	+ 370	− 370
Working load P = 11,000 lb each	−1010	+1010	+1790	−1790
Total	−1340	− 460	+ 90	−1890

* + for tension, − for compression.

To compute the cracking load, assume modulus of rupture at the average value of 820 psi. Then additional stress of 820 − 90 = 730 psi is required for cracking over the center support, which corresponds to additional loads P given by

$$\frac{4.63\,P \times 12 \times 8}{2730} = 730$$

$$P = 4500 \text{ lb each}$$

Total P required for cracking is hence 11,000 + 4,500 = 15,500 lb each. For cracking under the load, still following the elastic theory, P required to produce a stress of 820 + 460 = 1280 psi is given by

$$\frac{2.61\,P \times 12 \times 8}{2730} = 1280$$

$$P = 13{,}900 \text{ lb each}$$

Total P for cracking under the load is thus 11,000 + 13,900 = 24,900 lb each. Since the center support would have already cracked, more moment would be thrown to the points under the loads than indicated by the elastic theory, and the theoretical cracking load should be somewhat smaller than 24,900 lb.

The above computed values hold good for both beams A and B, since the commencement of cracking should be only slightly affected by the presence of additional steel bars.

COMPUTATION FOR ULTIMATE STRENGTH UNDER STATIC LOADS

In contrast to the cracking loads which were computed by the elastic theory, the ultimate strength must be estimated by plastic theories. Two plasticities are involved in a continuous prestressed structure: plastic theory for the strength of the critical sections, and plastic hinge theory for the strength of the beam as a whole. First, the ultimate flexural strength of the two critical moment sections are computed on the following assumptions:*

1. Plane sections remain plane up to the ultimate load.
2. Ultimate unit strain of extreme fiber of concrete = 0.0034.
3. Ultimate stress block in concrete is rectangular and averages 70 percent of the tested cube strength, or 70 percent × 8280 = 5800 psi.

*"Ultimate Strength of Prestressed Concrete Beams Failing in Flexure," *Bulletin*, Engineering Experiment Station, University of Illinois, 1953.

Under these assumptions and using the stress-strain curve of steel as shown in Fig. 5, a method of trial and error can be followed to determine the stress developed in the steel together with its acting lever arm at the ultimate load, thus giving a rather accurate estimate of the ultimate moment in spite of some inaccuracies involved in the above three assumptions. This method will be first applied to the section over the center support. From preliminary computations, it is seen that the steel wires will be stressed to about 90 percent of its ultimate strength or 90 percent $\times$ 256,000 = 230,000 psi which means a total tension in the wires of 0.963 $\times$ 230,000 = 221,000 lb. The total compression in concrete must be also 221,000 lb. The depth of the compressive stress block will be: 221,000/(5800 $\times$ 8) = 4.76 in.

(Note from Fig. 9 that the measured depth of compression was only 3.5 in. before failure.)

For this location of the ultimate neutral axis and an ultimate concrete strain of 0.0034, the ultimate strain in the wires can be computed as in Fig. 10(a): 0.0034 $\times$ 6.70/4.76 = 0.0048.

The strain in the steel due to the effective prestress of 120,000 psi is 120,000/29,000,000 = 0.0041. Hence the total strain in the steel is 0.0048 + 0.0041 = 0.0089. From Fig. 5, the stress in the wires corresponding to a strain of 0.0089 is 225,000 psi, which is quite close to the assumed value of 230,000 psi, so that no revision of the calculations is necessary and the ultimate stress in the wires can be estimated to be 225,000 psi.

The center of compression in the concrete can be assumed to be at 4.76/2 = 2.38 in. from the extreme fiber (this conservative assumption was made be-

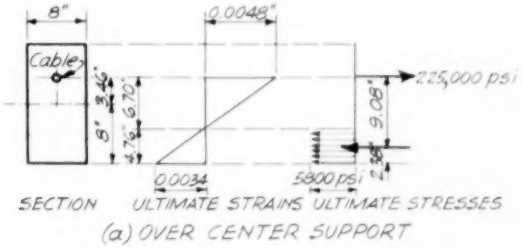


Fig. 10—Ultimate strength of sections, beam A

cause of its simplicity), and the lever arm for the tension is obtained as 9.08 in. The ultimate resisting moment is thus

$$0.963 \times 225{,}000 \times 9.08/12 = 165 \text{ kip-ft}$$

By similar computation, the stress developed in the steel under the load can be computed to be 218,000 psi at failure, and the resisting moment given by Fig. 10(b) as

$$0.963 \times 218{,}000 \times 7.69/12 = 134 \text{ kip-ft}$$

If full plastic hinges were obtained at the two critical sections, we would have a moment diagram as shown in Fig. 11(a) for the left span of the beam. Corresponding to this moment diagram the shear diagram and the loading diagram can be obtained from statics [Fig. 11(b) and (c)] and the value of P computed to be 41.5 kips. The measured breaking load of $P = 39.2$ kips indicated that full plastic action was not obtained, which fact was also borne out by the measured reactions as discussed later.

For beam B, a little investigation will show that the mild steel bars were stressed to the yield point of 45,500 psi at the ultimate load. The area of the two bars being 0.476 sq in., the total tension in the bars can be computed as $0.476 \times 45.5 = 21.6$ kips.

By a procedure similar to the above, and referring to Fig. 12, the ultimate moments can be computed to be, for the section over the center support

$$\frac{21.6 \times 12.22 + 212 \times 8.96}{12} = 180 \text{ kip-ft}$$

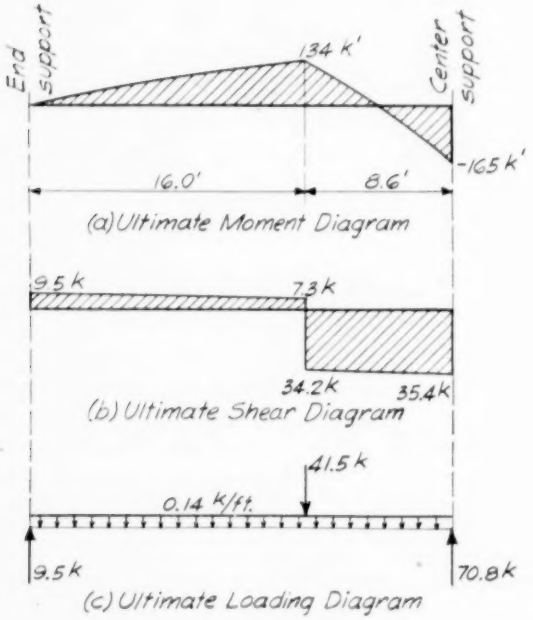


Fig. 11—Computation for ultimate load, beam A

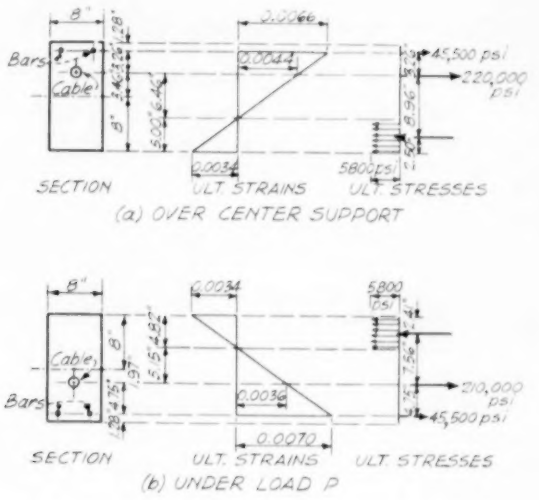


Fig. 12—Ultimate strength of sections, beam B

and for the section under the load

$$\frac{21.6 \times 12.31 + 202 \times 7.56}{12} = 148 \text{ kip-ft}$$

Again assuming full plastic hinge action, the load P that can be carried by the beam is computed by a procedure similar to Fig. 11. This yields $P = 45.8$ kips, which happened to check exactly with the measured value. It must be pointed out that 100 percent plastic hinge action actually was not realized in this beam, as shown by the reaction measurements (Fig. 13), while the actual ultimate flexural strength is believed to be higher than computed by the above method. Hence it is possible that the two errors neutralize each other and the computed value agrees exactly with the measured.

COMPUTATION OF CRACKING AND ULTIMATE STRENGTHS UNDER REPEATED LOADS

The fatigue flexural tensile strength of concrete is generally placed at about 60 percent of the static modulus of rupture. Since the number of load repetitions in these tests was not excessive up to cracking, it would be appropriate to assume that the strength at cracking was 80 percent of the static value, or 80 percent $\times$ 820 = 650 psi. For cracking over the center support, a stress due to live load of 650 − 90 = 560 psi was required, thus

$$\frac{4.63\,P \times 12 \times 8}{2730} = 560$$

$$P = 3440 \text{ lb each}$$

The total load at cracking is hence computed to be $P = 11{,}000 + 3440 = 14{,}440$ lb each. The measured values were $P = 15{,}800$ lb for beam C and 17,600 lb for beam D, even slightly higher than beams A and B under static loads only. This cannot be explained, although it might be attributed to

variation in modulus of rupture and to the smaller anchorage losses in beams C and D. Cracking in beam C was observed under static test while that in beam D occurred during pulsation loadings.

For cracking under the load, a stress of 650 + 460 = 1110 psi is required, which corresponds to an additional P given by

$$\frac{2.61\,P \times 12 \times 8}{2730} = 1110$$

$$P = 12{,}100 \text{ lb each}$$

and the total P should be 11,000 + 12,100 = 23,100 lb each. Since cracking had already occurred at the center support, partial plastic action was expected, and P should be somewhat smaller than the 23,100 lb given by the elastic theory. The actual measured values were 22,400 lb for beam C and 23,100 lb for beam D.

Ultimate strength under fatigue loads is more difficult to predict. Previous fatigue tests on similar wires showed that failure at points of gripping occurred for stress ranges from 106 to 142 ksi or 95 to 149 ksi.* Other fatigue tests on prestressed beams indicated that failure occurred in the wires over the separators when the average stress in the wires was about 150 ksi.† If it is assumed that failure started in the wires at a stress of 150 ksi, the corresponding load P can be computed as follows.

For approximation, assume a rectangular stress block in the concrete at an average stress of 50 percent the cube strength, *i.e.*, 50 percent × 8200 = 4100 psi; the depth of compressive stress over the center support of beam C was

$$\frac{150{,}000 \times 0.963}{8 \times 4100} = 4.4 \text{ in.}$$

and the lever arm for the tension in the wires was 9.26 in.; resisting moment was 150 × 0.963 × 9.26/12 = 111 kip-ft. Applying the elastic theory

$$4.63\,P = 111 - 10.5$$

$$P = 21.8 \text{ kips}$$

For partial plastic action P should be higher than this. The measured value was P = 24.2 kips.

For beam D, the tension in the bars was again 21.6 kips. The depth of compressive block was

$$\frac{21{,}600 + 0.963 \times 150{,}000}{8 \times 4100} = 5.06 \text{ in.}$$

The resisting lever arm was 12.9 for the bars and 8.93 in. for the wires, hence the resisting moment was

$$21.6 \times 12.19/12 + 144 \times 8.93/12 = 22 + 107 = 129 \text{ kip-ft}$$

Again applying the elastic theory,

$$4.63\,P = 129 - 10.5$$

$$P = 25.6 \text{ kips}$$

*Soete, W., and Vancrombrugge, R., "The Resistance of Fatigue of Wires Used in Prestressed Concrete," *Library Translation* No. 25, Cement and Concrete Assn., London, 1951.

†(a) Billington, D. P., "The Dynamic Testing of Self-Anchorage in a Prestressed Concrete Beam," *Precontrainte* (Brussels), No. 2, 1952, p. 84.

(b) Magnel, G., *Prestressed Concrete*, 3rd Edition, McGraw-Hill Book Co., Inc., New York, N. Y., 1954, pp. 156-163.

The measured value was 28.6 kips, which perhaps can be again explained by the partial plasticity of the beam.

END REACTIONS

The accuracy of the elastic or plastic theory was assessed by the measured and computed reactions at the ends of the beams. First of all, the concordancy of the cables was checked by the dead load reactions, whose measured values agreed well with the elastic theory, as shown in Table 8.

TABLE 8—MEASURED AND COMPUTED REACTIONS FOR DEAD LOAD

	Beam A	Beam B	Beam C	Beam D
Measured, lb	1430	1380	1470	1430
Computed, lb	1420	1410	1450	1410

For live load, the measured values are plotted in Fig. 13 for beams A and B. Two theoretical lines are also plotted, one by the elastic theory, another by the plastic hinge theory, assuming the moments developed at each hinged section to be proportional to its ultimate moment capacity. It is seen that before the appearance of cracks, the reactions followed the elastic theory as closely as can be measured. After the commencement of cracks, the reactions deviated gradually toward the plastic theory. Since the beam failed in compression in the concrete, complete plastic hinge did not develop, and the behavior of the beam was only partly plastic even up to failure. There was a slight difference between the two beams near the ultimate load; the greater elasticity of beam B might be attributed to the presence of mild steel bars.

The static reactions to beams C and D, measured after every 500,000 cycles of loading, are plotted in Fig. 14. They followed essentially the same pattern as those for beams A and B. There was, however, some plastic action for the lower range of loads after the fatigue tests. The deviations were small and could have been errors in the gage readings. Beam D behaved similar to beam C except that there was slightly less plasticity near the ultimate range.

STRAIN GAGE READINGS

Curves for the strain gage readings showed similar shapes, typical ones of which are plotted in Fig. 15 and 16. For beams A and B, several curves deviated from a straight line almost as soon as the fibers were under tension. The modulus of elasticity also differed greatly for the gages. Tests made on a plain concrete beam also showed nonuniformity of tensile strains. It is believed that strains vary extensively for concrete under tension so that local measurements with a gage length of only 1 in. cannot be used as representative of the beam.

For the beams under fatigue tests, on the other hand, the strain curves all continue straight right up to cracking. This cannot be well explained,

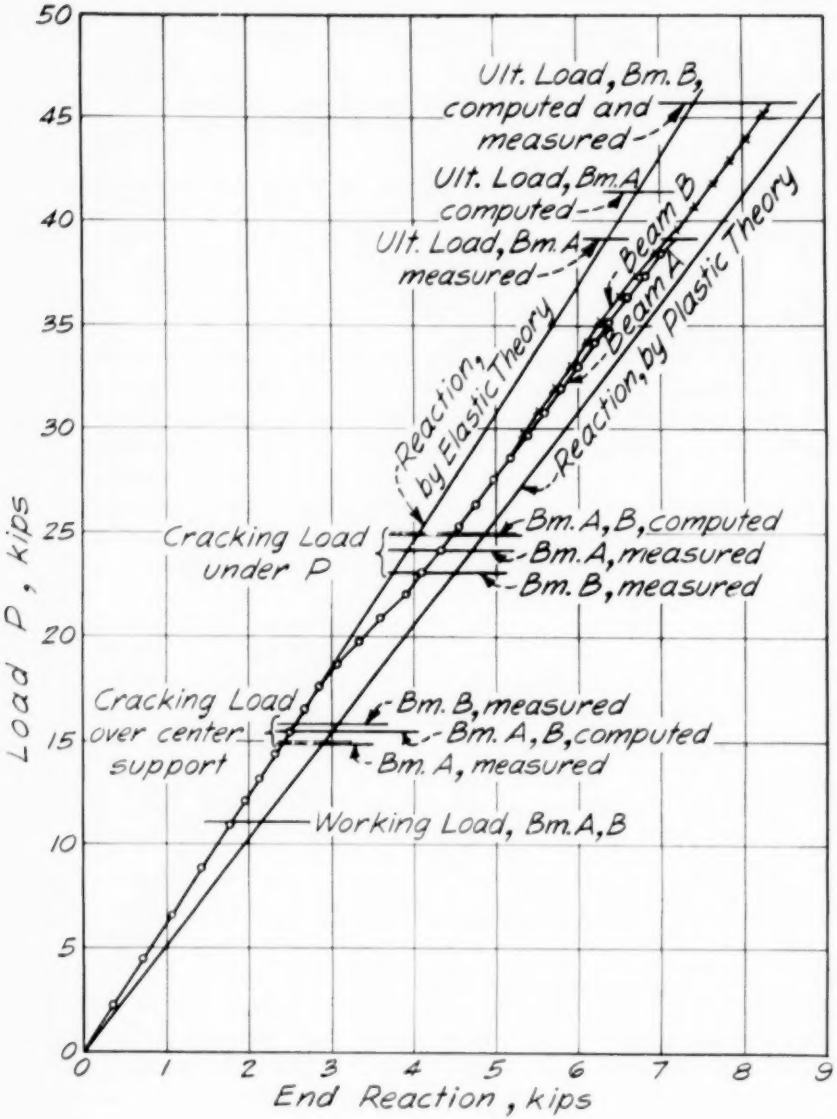


Fig. 13—End reactions, beams A and B—measured versus computed

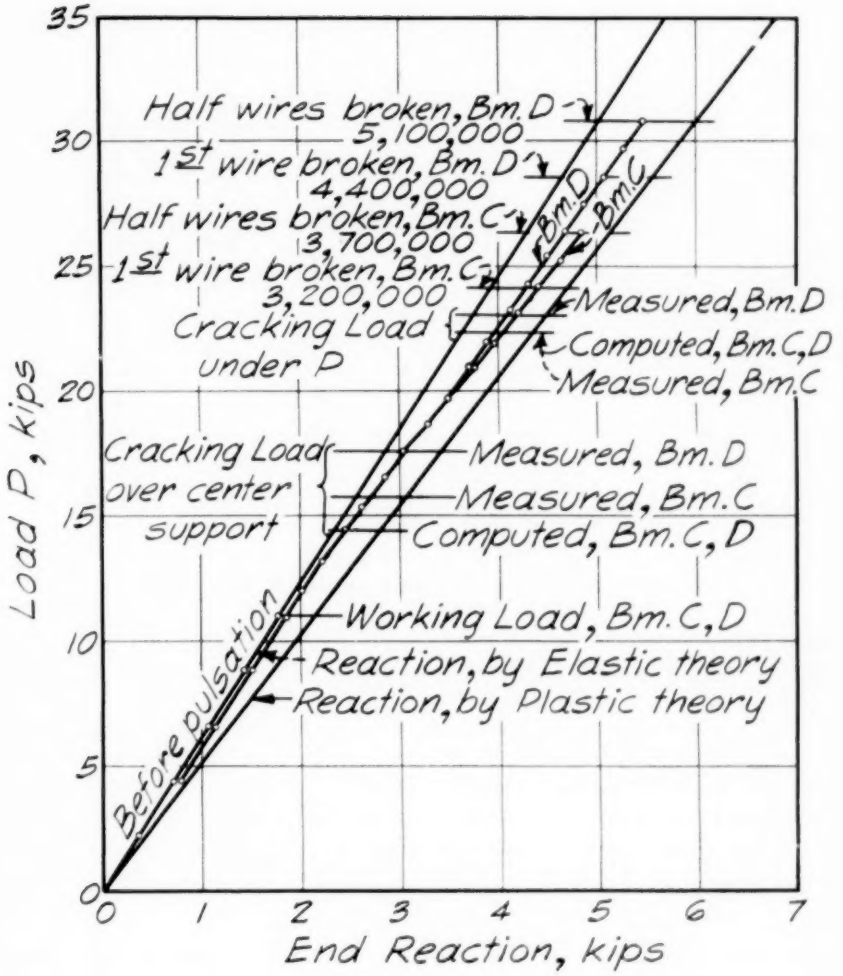


Fig. 14—End reactions, beams C and D—measured versus computed

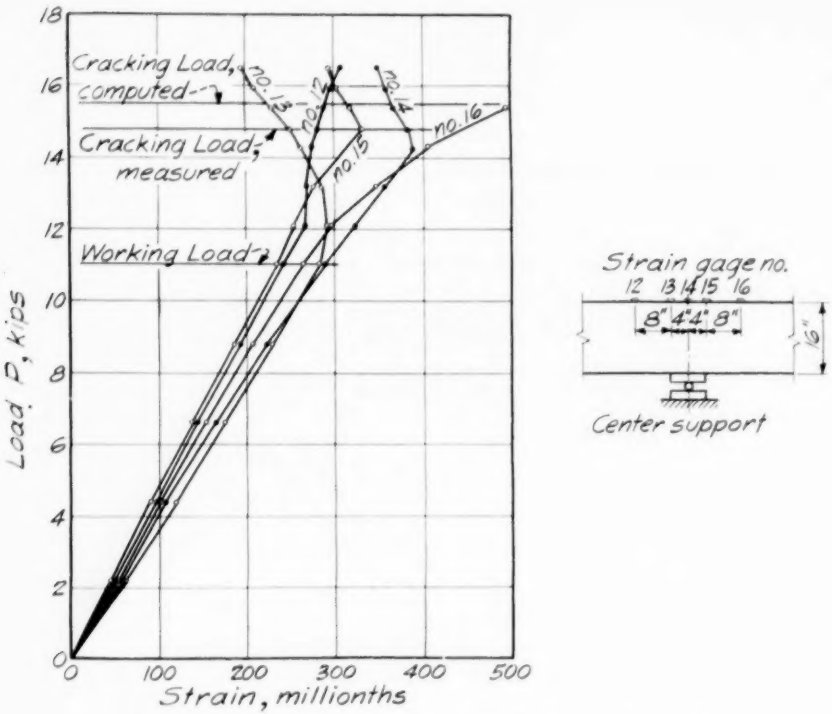


Fig. 15—Typical strain gage readings over center support, beam A

but it is guessed that the prestress in beams C and D could have been higher than assumed so that the fibers were actually under some compression when computed to be under tension.

In all cases, however, as soon as cracking commenced near the gages, the strain readings dropped sharply. If the crack occurred right through the gage, the readings jumped sharply upward. As an auxiliary test, several strain gages were placed directly over cracks. Even while under compression, the strains across these cracks due to de-compression were found to be greater than normal.

DEFLECTIONS

Midspan deflections of the beams averaged 0.090 in. upward due to prestressing, gradually increasing to 0.110 in. in about 3 days. According to the elastic theory, for a modulus of elasticity of 5,500,000 psi, the deflection should have been only 0.065 in. It is believed that some creep took place during the process of prestressing, thus increasing the measured deflections.

Deflection curves under loading were plotted for beams A and B (Fig. 17). Within the working load and up to cracking, there was no noticeable difference between the deflections of the two beams. But as the ultimate load was

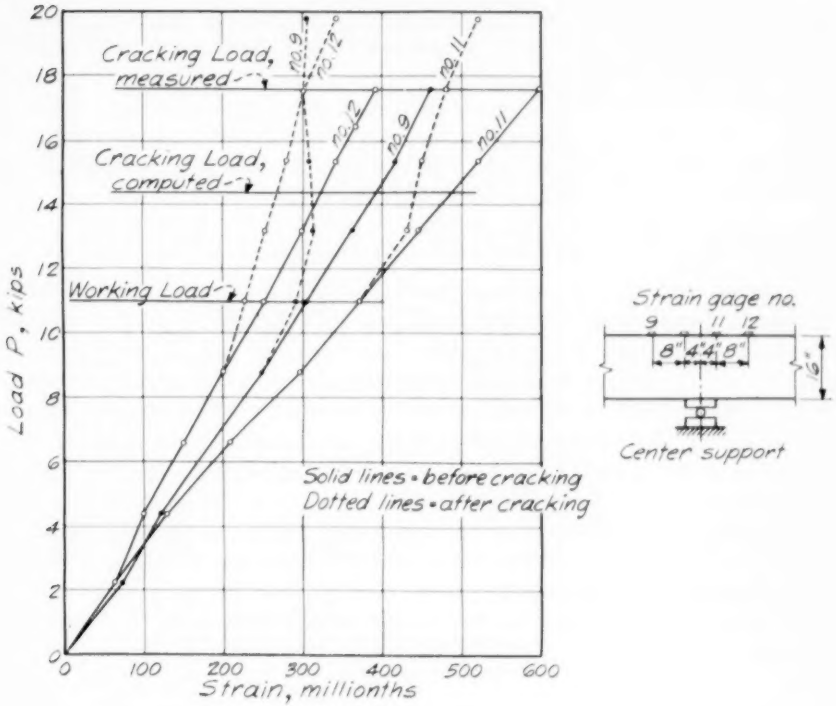


Fig. 16—Typical strain gage readings over center support, beam D

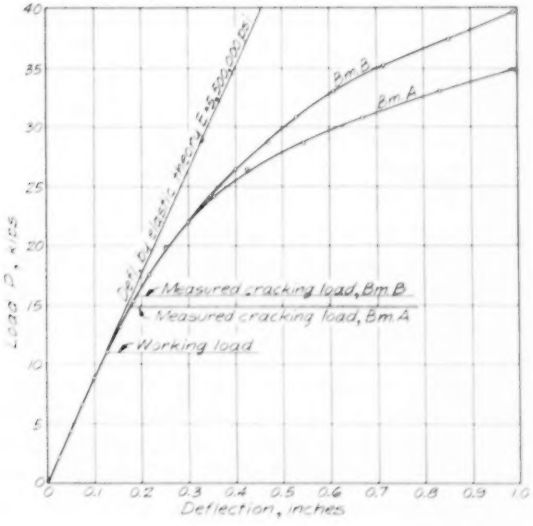


Fig. 17—Midspan deflection, beams A and B

approached, deflection of beam B was smaller than beam A. This could be explained by the presence of the mild steel bars, which were more effective near the ultimate load.

Beams C and D under repeated loads showed deflections similar to those of beams A and B. There seemed to have been a slight increase in the deflections after each range of pulsation, in the order of 3 to 5 percent. This can perhaps be attributed to either creep or fatigue effect.

CONCLUSIONS AND DISCUSSION

Although only four beams were tested, due to the accuracy of instrumentation and the large size of the beams, certain conclusions can be definitely advanced. Basic principles for computing cracking and ultimate strength under static loads can be proposed based on these tests. Under repeated loads, however, only the general nature of the ultimate strength can be suggested. Test results are discussed below.

1. Within the working load, the beam behaved according to the classical elastic theory as closely as can be measured, when subjected to either static or repeated load. The moment in the beams can be computed within an accuracy of 1 percent, as shown by the measured reactions. The instantaneous deflections can be computed by elastic theory, using the modulus of elasticity measured by compression tests, within an accuracy of about 10 percent. Deflection of the beam related to loads remained linear up to the working loads.

2. Between the working and the cracking load, the beams still behaved elastically in so far as their reactions and moments were concerned, but the deflections increased slightly faster than the load, which fact was attributed to the nonlinear stress-strain relation of concrete. Under repeated loads up to cracking, the reactions and moments deviated from the values given by elastic theory by about 3 to 5 percent. For the prediction of the commencement of cracking, the elastic theory was almost exact for the beams under static tests, but was conservative by about 16 percent for the fatigue tests.

3. For beams under static loads only, local tensile strains exhibited nonlinear relations with load almost as soon as tension started in the fibers. For beams under repeated loads, the stress-strain relation responded linearly right up to cracking. No reason can be advanced for this phenomena.

4. Due to plastic action in a continuous beam after cracking the appearance of cracks under the second hinges may be slightly sooner than given by elastic theory, but can be quite accurately computed if the plastic action is taken into consideration.

5. For over-reinforced beams, complete plastic hinge action under static load cannot be expected. The percentage of plastic action will depend upon the mode of failure. The ultimate static load on a continuous beam can be fairly well estimated by simple rational methods.

6. For beams under repeated loads, the nature of failure was entirely

different. Fatigue failure of the wires at the separators took place under repeated heavy loads producing serious cracks at points of maximum moments. Nominal average stress in the wires was about 25 percent higher than the effective prestress.

7. The measured factor of safety of these beams, considering live load only, was:

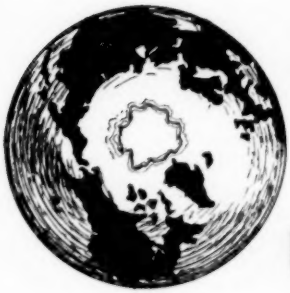
Beam	A	B	C	D
Loading	Static	Static	Repeated	Repeated
Mild steel bars	Without	With	Without	With
Factor of safety against cracking	1.35	1.44	1.44	1.60
Factor of safety against rupture	3.54	4.15	2.20	2.60

For beams C and D, the factor listed above was for the breaking of the first wire, after which the beam still stood many cycles of heavier loads, and carried from 3.0 to 3.7 times the working under the final static test after half of the wires had broken.

8. The presence of mild steel bars increased the cracking load only slightly, by 5 to 10 percent, but they distributed the cracks and limited their extension. The ultimate loads under both static and repeated loads were increased by 18 percent for a small amount of bars strategically positioned.

ACKNOWLEDGMENT

The guidance and advice on these tests given by Prof. G. Magnel has been most valuable. The help rendered by G. Huyghe, H. Lambotte, and Philemon Bouderez in the laboratory is also gratefully acknowledged.



Letters from Readers*

BY WAY OF SYNOPSIS

E. L. Howard and W. E. Moulton discuss the controls and test data accumulated on 15,000 cu yd of Haydite concrete placed in a 27-story office building.

Close Control of 15,000 cu yd of Haydite Concrete for a 27-Story Building (LR 51-24)

In downtown San Francisco engineers have planned and built an office building 210 ft high with 458,000 cu ft of floor space. The steel frame structure required 22,000 cu yd of concrete for foundations, walls, and floors. Twenty-five of the 27 stories were placed using 15,000 cu yd of Haydite concrete. This report concerns the controls used and the test data accumulated on the lightweight concrete portion of the job.

Batch plant operations

Before each day's placement, the Haydite aggregates were transferred by conveyor belt from plant surge bins to the bunkers over the weigh hoppers. Haydite arrived in a semi-dry condition and was brought to saturation by the water-spray system installed over conveyor belts (Fig. 1).

The batching plant is a two-unit premix plant. One unit of the plant produces only normal weight concrete. The unit used for Haydite makes normal weight concrete on days Haydite concrete is not produced. The aggregates are stored in five bins above each weigh hopper and four silos for cement are available. All aggregates are weighed automatically in one hopper with multi-beam scales. Cement is weighed separately; water and air-entraining agent are measured by volume. All concrete is premixed in 3-cu yd tilting mixers. These mixers (one in each batching unit) dump into holding hoppers from which the transit truck draws its load.

The Haydite placements, averaging 300 cu yd, were made two or three times a week. Aggregates previously saturated were transferred to the batch plant bunkers as needed.

The transit trucks, with mixers turning at 3 rpm hauled 6 cu yd of concrete. The trip through city traffic to the job site required 15-20 min. Street parking at the job site was not permitted because of traffic congestion. Trucks had to be timed perfectly

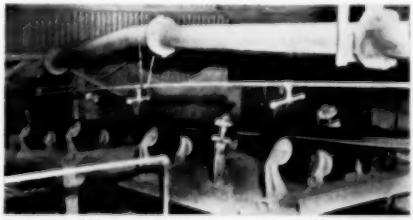


Fig. 1—Water-spray system installed over a conveyor belt in batch plant to saturate Haydite aggregate

*A part of copyrighted Journal of the American Concrete Institute, V. 26, No. 10, June 1955, *Proceedings* V. 51. Separate prints of the entire Letters from Readers section are available at 35 cents each. Address 18263 W. McNichols Rd., Detroit 19, Mich.

to keep concreting going smoothly. A truck waiting station, manned by a dispatcher with a telephone, was established four blocks from the job, and trucks waited their turn to enter the construction area.

The first few placements were marked with a considerable backlog of loaded trucks at the waiting station. A delayed load lost slump and was the source of much irritation to inspectors and concrete placing crews. After a half-dozen placements, when traffic control and job unloading became more efficient, only a rare breakdown of handling equipment disturbed the smooth flow of concrete.

An inspector at the batching plant, with close cooperation of the batch man, observed each batch, making adjustments to maintain uniformity of quality. Of the 53 placements, batch weight adjustments were required during 37. From one to four adjustments of batch weights were necessary during each of these. At the job site a service man kept in touch by telephone with the batch plant and the waiting station. His reports kept the dispatcher and the plant inspector informed about the concrete on the job.

Materials

Haydite aggregates were sampled on arrival at the batching plant. Gradings of Haydite are shown as percentages passing a given sieve in Table 1. Because the specific gravity varies with the size fraction, these sieve analyses do not give a true picture of the relative distribution of particle size.

TABLE 1—AVERAGE VALUES OF TEST DATA ON HAYDITE CONCRETE AGGREGATES

U. S. sieves	Aggregate, percent passing			
	¾-in.	⅜-in.	Fines	Natural sand
¾-in.	100			
½-in.	64			
⅜-in.	20	100		
¼-in.	7	76		
No. 4		47	100	100
No. 8		13	82	87
No. 16		10	48	63
No. 30		5	29	42
No. 50		2	18	17
No. 100			11	4
No. 200			7	1
Specific gravity	1.35	1.66	1.82	2.66
Absorption, percent	14.1	16.0	12.0	1.70
Unit weight, lb per cu ft loose	39	42	52	90
Moisture, percent	3.85	4.45	4.44	6.00

These materials were received in three sizes: ¾ to ⅜ in., ⅜ to ⅛ in., and minus ⅛ in. Specific gravities for design and control purposes were determined from the delivered materials, the larger material having the lower specific gravity of 1.35 and increasing approximately 0.25 with each smaller size. The Haydite aggregates were brought to saturation before use in the concrete, which meant an increase in water content of about 10 percent over the moisture on delivery. The success of this operation greatly effected the control of the concrete yield and consistency.

Natural sand fineness modulus was uniformly 2.87 and the moistures varied but slightly from 6 percent. The air-entraining agent was added to the water discharge line through an automatic dispenser. City water was used.

The specification requirements were for concrete weighing no more than 100 lb per cu ft dry and having a compressive strength of 2750 psi at 28 days. A maximum slump of 5 in. was required.

Proportioning and control

Proportioning was according to "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)." This method produced a workable and satisfactory concrete in all respects. Table 2 details the mix proportions that were used throughout the job. Adjustments made at the batch plant as materials varied insured delivery of the designed mix. Added water varied from 25 to 41 gal. per cu yd, and averaged 35 gal.

Samples for the unit weight of hardened concrete were taken at the job site; all other samples were taken at the batch plant. For the purposes of control the unit weights were of great importance. They furnished the data for yield calculations and building weight estimates. Unit weights of hardened

TABLE 2—MIX PROPORTIONS FOR HAYDITE CONCRETE

Materials	Lb for 1 cu yd concrete	Percent of total volume
¾-in. Haydite	390 (SSD)	35.0
⅜-in. Haydite	440 (SSD)	
Fines, Haydite	220 (SSD)	7.8
Natural sand	960 (SSD)	21.6
Cement	620	11.7
Water	333	19.8
Air-entraining agent	4 oz	4.1 percent air

TABLE 3—HAYDITE CONCRETE TEST DATA

Test	Average	Standard deviation	Number tests
Unit wt. fresh concrete	109.5 lb per cu ft	5.70 lb	182
Unit wt. hardened concrete, wet	111.4 lb per cu ft	4.15 lb	48
Unit wt. hardened concrete, dry	96.6 lb per cu ft	3.70 lb	48
Concrete yield per cu yd	27.3 cu ft	0.71 cu ft	182
Air percent by volume	6.7 percent	1.7 percent	22
Kelly ball penetration	5 in.	1.2 in.	182

concrete averaged, when saturated, slightly more than fresh concrete but after drying met the specified weight requirement of 100 lb per cu ft (see Table 3).

Air content was very uniform as measured by the pressure meter. Only one very high air content was noted, and this on a day the Haydite aggregate was very dry. The measured air content of 6.7 percent may not have been all entrained air. Entrapped air varied with the absorption of the Haydite and accounted for some of the small variance in air content. The air entrainment gave workability to the concrete and prevented segregation. Consistency was controlled by the Kelly ball test (Table 3). The average penetration of 5 in. is equivalent to about a 4½-in. slump. The standard 30-lb ball was used. On other jobs a 20-lb ball has been used with lightweight concrete.

Haydite moistures shown in Table 4 were taken at the weigh hopper. They include all the water in the aggregate. When the total moisture is greater than the absorption a uniform concrete can be supplied. When the moisture is less than absorption some of the mixing water is taken up by aggregate. The deviation in moisture became the biggest problem in quality control. Moisture changes were most serious after a delay or postponement of placing. Aggregate previously saturated had opportunity to lose a portion of its moisture before concreting was resumed.

TABLE 4—ABSORPTION AND MOISTURE OF HAYDITE

	¾-in.	⅜-in.	Fines
Average absorption, percent	14.1	16.0	12.0
Average total moisture at weigh hopper, percent	15.8	17.2	14.0
Average free moisture, percent	1.7	1.2	2.0
Free moisture limits, percent	0–6.7	0–6.2	0–7.0

TABLE 5—COMPRESSIVE STRENGTHS OF HAYDITE CONCRETE

Average psi—28 days	3410
Standard deviation, psi	349
Number of tests (average of three cylinders each)	68

The compressive strength data shown in Table 5 are those tests made by Pacific Coast Aggregates, Inc., and were taken on each day's concreting. These cylinders showed an average compressive strength of 3410 psi and evidence concrete 600 psi more than the required compressive strength. The percent of variation for this group of compressive tests was 10 percent.

Tests made during each placement were the basis for mix control adjustments. From the unit weight tests of fresh concrete the yield of the nominal cubic yard can be calculated. The yield averaged 1.1 percent over the designed 27 cu ft; the yield efficiency can be credited to the careful control exercised at the mixing plant. Mix adjustments were continually made to maintain the actual volume at the nominal cubic yard. Estimations of batch volume were made at the calibrated holding hoppers at the batch plant and on the job about every 100 cu yd.

For the whole job 15,247.5 nominal cu yd were delivered. By the calculated yield 15,431 cu yd were actually batched. This is 183.15 cu yd (1.2 percent) over the nominal yardage. Only 8453 cu yd of concrete were checked by volume measurements in the job receiving hopper. In this case the percent accuracy of yield was 0.1 percent.

Conclusions

1. Those close to this job are convinced that good lightweight concrete is made when the aggregates are saturated. Even with proper mix adjustments, uniformity of consistency and yield could not be obtained with dry aggregates.

2. The plant inspector must have authority to adjust mixes as the need arises. On this job some 15 combinations of weights were furnished the batch man. As the concrete visibly changed the appropriate batch weights were used until accurate checks and adjustments were made.

3. The Kelly ball is an efficient tool to measure and control consistency of lightweight concrete.

4. Air entrainment greatly improves workability and prevents segregation.

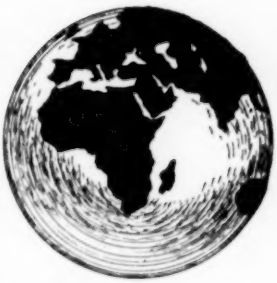
5. Compressive strengths can be obtained comparable to sand and gravel aggregates.

6. The standard ACI 613-54 water-cement ratio tables for air-entrained concrete can be used safely for mix proportioning, but better than average inspection is necessary to keep Haydite concrete within specification limits. Extra care must be taken with the control of moisture of aggregates, the batching, mixing, and delivery of the concrete.

Using careful inspection and control as described in this report the problems expected with Haydite concrete become routine operation.

E. L. Howard and W. E. Moulton, Pacific Coast Aggregates, Inc., San Francisco, Calif.

Current Reviews*



of Significant Contributions in Foreign and Domestic Publications

Construction

Lining of galleries with pre-tensioned concrete (Revestimiento de galerias con "hormigon pretensado")

ALOIS KIESER, *Informes de la Construccion* (Madrid), V. 7, No. 64, Oct. 1954, Item 579-4, 6 pp.

Study of a new procedure for lining galleries which consists of a primary ring of concrete formed directly against the tunnel walls followed by another concentric lining of precast elements. In the space between the two linings, mortar is injected under pressure to constitute a tightening or pre-tensioning chamber. The procedure, used on ten projects in Austria, Germany, and Italy, is said to have advantages over ordinary methods of tunnel lining.

Shepherd's Bush garage

The Engineer (London), V. 198, No. 5153, Oct. 29, 1954, pp. 592-593

Reviewed by ARON L. MIRSKY

Garage with a capacity of 123 buses is situated on an irregularly-shaped site, occupied in part by an existing garage. Requirement of maintaining garage in operation necessitated stage construction. Garage is characterized by some unusual concrete construction, including a bowstring girder of 132 ft span and 26.3 ft rise, several smaller bowstring girders, and long (108 ft maximum) box beams. All joints between principal elements were carefully articulated; girders are supported by hinged connections at one end and roller bearings at the other.

Collapsible steel forms for the diversion tunnel of the Tauern power works Glockner Kaprun (Zusammenfaltbare Stahlschaltung fuer den Moellstollen des Tauernkraftwerkes Glockner Kaprun)

FRIEDRICH MESCHAN, *Der Bauingenieur* (Berlin), V. 29, No. 2, Feb. 1954, pp. 50-52

Reviewed by ARON L. MIRSKY

Four-kilometer length of diversion tunnel of this project (see "Current Reviews," ACI JOURNAL, June 1953, *Proc.* V. 49, p. 979) was in rock. Hole was 3.4 m in diameter, net diameter of lined tunnel was 3.0 m. Article describes forms and concreting train.

Thin-shelled dome roof for boxing stadium

KNUD KIELER, *Indian Concrete Journal* (Bombay), V. 28, No. 7, July 1954, pp. 259-262

Boxing stadium roof in Bangkok, Thailand, consists of a dome and a flat roof along its edge. The dome is carried by 16 columns. On top of the columns is a heavy, circular ring beam. The dome proper is supported by 64 inclined slender columns arranged in a zig-zag pattern similar to the supports for cooling towers at many European power stations. The dome is spherical, with a radius of 44.7 m and a rise of 5.7 m. The shell is 8 cm thick, increasing at the edges to match the thickness of the inclined columns. Underside of the dome is lined with 3.5 cm thick slabs of "wood wool concrete," which served as bottom forming for the dome during casting.

Since the owners held boxing matches once a week during construction, an intricate and

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ingenious centering and scaffolding arrangement was devised. Since a great number of posts supporting the formwork would obstruct the view of the boxing ring and reduce seating capacity, a central tower and trusses spanning to the ring beam supported the formwork. The tower was supported on four reinforced concrete columns placed at the corners of the boxing platform.

Sixteen-story skyscraper of no-fines concrete for the student project at the Stuttgart Technical Institute (Sechzehngeschossiges Hochhaus in Schuettbeton fuer das Studentenwek an der Technischen Hochschule Stuttgart)

K. Deininger and F. Leonhardt, *Zeitschrift VDI* (Duesseldorf), V. 96, No. 24, Aug. 21, 1954, pp. 814-816

Reviewed by Aron L. Mirsky

Sixteen-story dormitory of no-fines concrete, using brick rubble as aggregate, was built by special permission; previously such construction had been restricted to structures up to five stories in height. Outer walls were 37.5 cm thick, inner walls 25 cm; controlling factor was heat and noise insulation value. Construction took one week per story, including floor slab.

Sulfuric acid plant. Anhydrite process at Widnes includes cement production

Engineering (London), V. 178, No. 4633, Nov. 12, 1954, pp. 640-641
The Engineer (London), V. 198, No. 5158, Dec. 3, 1954, pp. 770-772

Reviewed by Aron L. Mirsky

Descriptions of new plant designed to produce 148,000 tons of sulfuric acid and 132,000 tons of cement clinker per year. Anhydrite process is rendered economically feasible only by reason of cement made from clinker resulting from process; adjacent plant, designed to produce cement with 5 percent gypsum added, is interlocked with anhydrite plant to assure proper flow of materials and sequence of operations.

United Kingdom school incorporates new construction system

British Constructional Engineer (London), V. 5, No. 2, Sept. 1954, pp. 42-44

The "Intergrid" method, used in construction of a secondary technical school, consists of a light prestressed concrete frame combined with standard structural units. Elements are precast in the factory under close control and assembled on the site. System is designed on 3 ft 4 in. module. System makes fullest use of precasting and prestressing. Design and construction details cover columns, column heads, beams, floor and roof slabs, stairs, wall panels, surface finishes, and eaves. (See also "Current Reviews," ACI Journal, Feb. 1955, *Proc.* V. 51, p. 585.)

Outstanding example of modern concrete construction

British Constructional Engineer (London), V. 5, No. 3, Dec. 1954, pp. 22-25

The new printing plant for the Bank of England is an outstanding combination of in-situ, precast, prestressed, and shell construction. Building operations are described. (See also "Current Reviews," ACI Journal, Feb. 1955, *Proc.* V. 51, pp. 579 and 586.)

Construction of a laboratory building for Siemens & Halske A. G. at Munich (in German)

O. Schinzel, *Precontrainte* (Brussels), V. 4, No. 1, Jan.-June 1954, pp. 14-23

Author's Summary

This nine-story building has a span of 16 m without columns. The frames are statically indeterminate and the prestressing cables of the beams pass through the columns, which gives the frame action. The beams are 3.5 m center to center. Floors are of monolithic concrete; the superimposed load is 600 kg per sq m. Prestressing and construction operations are detailed.

Short electric tunnel kiln for building products in Uganda

Douglas Bond, *The Engineer* (London), V. 198, No. 5159, Dec. 10, 1954, pp. 809-813

Reviewed by Aron L. Mirsky

Design, construction, and operation of an electric kiln with a 100-ft effective length are described. (A brief description of the plant was published in same journal, V. 198, No. 5138, July 16, 1954, p. 98; see "Current Reviews," ACI Journal, Jan. 1955, *Proc.* V. 51, p. 482.)

Dams

Conon Valley hydroelectric scheme

Engineering (London), V. 178, No. 4633, Nov. 12, 1954, pp. 629-633

Reviewed by Aron L. Mirsky

Description of Loch Luichart dam (including a fish ladder) and Meig and Loch Luichart tunnels, parts of the second stage

of a hydroelectric development in the north of Scotland which will have an estimated annual capacity of 440 million kilowatt hr.

Oberhasli power schemes

Engineering (London), V. 179, No. 4641, Jan. 7, 1955, pp. 13-16; No. 4642, Jan. 14, 1955, pp. 50-53

Reviewed by ARON L. MIRSKY

Describes development scheme for the River Aare in the Haslital Valley of Switzerland. First plant (Handek I) went into operation in 1932; fourth (Grimsel) in 1954. Total estimated annual production is 1230 million kilowatt hr.

Ullapool Hydroelectric scheme. Rural development in western Scotland

Engineering (London), V. 178, No. 4629, Oct. 15, 1954, pp. 495-496

Reviewed by ARON L. MIRSKY

Gravity dam across outlet of Loch Dubh is 242 ft long and 40 ft high, built of "Colcreted" local stone with a maximum size of 8 in. Intake dam, 241 ft long and 31 ft high, is of normal mass concrete.

Measurements on the main penstock of the Lech reservoir at Rosshaupten (Messungen am Hauptstollen des Lechspeichers Rosshaupten)

J. FROUNHOLZER, Bayerische Wasser-Kraftwerke-AG, Munich, 1953, 75 pp., 7 DM

Reviewed by ARON L. MIRSKY

This penstock, one of Europe's largest (with a diameter of 8.5 m), was constructed in 1951-1952. It was prestressed by injecting mortar under a pressure of 5 atmospheres, followed by grout under 8 atmospheres pressure, between two rings of concrete (the method of Kieser and Berger). Reported here are the results of tangential, radial, and axial strain measurements made with electroacoustic gages from July, 1951, to October, 1952, with a supplement to February, 1953. Results were highly favorable.

An abstract of this report, with conclusions, appeared over the name of F. TOELKE in *Zeitschrift VDI*, V. 96, No. 21, July 21, 1954, p. 729.

Cabril hydroelectric project (in Portuguese)

CARVALHO XEREZ, *Tecnica* (Lisbon), V. 29, No. 243, June 1954

Reviewed by J. LAGINHA SERAFIM

Describes the arrangement of the project which includes power plant, intake tower, two spillway tunnels, and the highest Portuguese arch dam (440 ft). Summarizes the studies for the design and the main features of the construction.

Design

Analysis and tests of a cylindrical shell roof model

BRUNO THURLIMANN and B. G. JOHNSTON, *Proceedings*, ASCE, V. 80, Apr. 1954, Separate No. 434, 30 pp., $0.50

AUTHORS' SUMMARY

Analytical expressions for the effective width of cylindrical shells are presented. These formulas are used for the analysis of a cylindrical shell roof acted upon by horizontal longitudinal forces. Also studied are the influences of foundation movements, temperature changes, and shrinkage.

A comparison is made between theoretical results and experimental measurements obtained from a model of an actual cylindrical shell roof constructed in the approximate ratio of 1 to 30. It is established that there is satisfactory agreement between the theoretical analysis and the test results.

Shell versus arch action in barrel shells

M. G. SALVADORI, *Proceedings*, ASCE, V. 81, Separate No. 653, Mar. 1955, 10 pp., $.50

In this paper the author has extended his analysis "Ribless Cylindrical Shells," ACI JOURNAL, Jan. 1955, *Proc.* V. 51, p. 457. As he points out, cylindrical shell roofs are usually designed as thin flexureless membranes stiffened by transverse ribs or arches. Transmission of loads by the thin shell to its stiffeners from which it hangs is defined as "shell action" while the behavior of the unstiffened shell resting on the foundation is called "arch action." This paper is an analysis of the combination condition, *i.e.*, when a cylindrical shell stiffened by transverse ribs is also supported by the foundation. In this case, of course, the loads are transmitted to the foundation by a combination of both arch action and shell action. The author has evaluated the bending moments created at the edge and the vertical load created by this arch action for a wide range of shell proportions, and for a particular case in which the shell is cylindrical, the cross section may vary, tangents at the edge are vertical, stiffeners are infinitely rigid, stiffener connection to the foun-

dation is infinitely rigid, and loads are uniformly distributed.

This paper should serve well to extend the area of application of thin shells for practical designers.

Strength of materials (Resistance des materiaux) —V. 2

P. Beteille. Edition Eyrolles, Paris, 1954, 224 pp. Reviewed by Alexander M. Turitzin

This second volume covers subjects which were not treated in the first volume. The latter was entirely devoted to beams, statically determinate and indeterminate. The first part of the second volume covers analyses of arches, such as single-hinged arches, two-hinged arches, three-hinged arches, fixed arches, symmetrical and unsymmetrical arches, arches with tie rods, and bowstring arches without diagonal web members, often called bowstring girders. Of particular interest, is the analysis of a three-hinged arch subject to horizontal earth pressure and a description of a method of stress control by means of hydraulic jacks employed during decentering of the concrete arch ribs. This subject of arch analysis which covers almost half of the volume, although written in a condensed manner, is perhaps the best part of the book.

The second part is a brief statement of the Hardy Cross method of moment distribution, and its application to rectangular frames. A method of solving frames by means of angle rotations is briefly given and then compared to the method of moment distribution. It is stated that the latter method is preferable to the former if the structure is subject to sidesway, and if the computer has a calculating machine at his disposal.

The third part is a brief treatment of the fundamental methods of analysis of statically determined trusses and the application of the Castigliano method of least work to the analysis of trusses which are statically indeterminate. The effects of joint rigidity and of secondary stresses are briefly explained.

The fourth part of the book deals with the derivation of Euler's critical buckling load formulas for different end conditions of columns. Also, the derivation for members subject to longitudinal compression and transverse bending, is presented.

In the fifth and the last part, the effect of prestressing on simple and continuous beams is determined and its effect on arches and cylindrical structures is discussed.

With the exception of the second portion, wherein the author presents two illustrative examples of the application of moment distribution and the method of angle rotations, there are no numerical problems in the book. The material is greatly condensed and should be useful as a reference book rather than a textbook. Most of the material presented is amply treated in textbooks on the theory of structures.

Materials

Pozzolanas and their use in concretes

K. M. Alexander, *Constructional Review* (Sydney), V. 27, No. 6, Oct. 1954, pp. 22-28

Discusses first the types of natural and artificial materials which are likely to be pozzolanic and considers methods for assessing pozzolanic efficiency. Then the extent to which the properties of portland cement mortars and concretes are modified by pozzolans is covered.

H. M. S. process solves substandard material problem for gravel firm

William M. Avery, *Pit and Quarry*, V. 46, No. 4, Oct. 1953, pp. 87-91, 122

To process out shale and soft-rock inclusions from what was otherwise a good sand and gravel deposit, heavy-media separation equipment was installed. Plant set-up is described.

Methods and results obtained by small-sized grain techniques used in mass and reinforced concrete (Wege und Erfolge der Feinstkorntechnik im Massen- und Stahlbetonbau)

J. Fritsch, *Zement-Kalk-Gips* (Wiesbaden), V. 7, No. 8, Aug. 1954, pp. 312-315 Reviewed by H. H. Werner

Technical equipment used hitherto on large building sites for washing and separating sand did bring about improvements in composition. However, such equipment is often inefficient and always removes some parts of the most valuable fine grade sand. A process developed in recent years permits classification of even the finest particles correctly into granulation groups which are later added, as has been previously done with coarse aggregates.

A graphical calculation method is given which provides the possibility of determining

the most favorable mixing ratio of three sand granulation groups. In so doing, the value of a grain mixture is judged according to the cement paste required under identical conditions to obtain first class compactability. This method appears to be quite promising with regard to its application in central concrete mixing installations.

Chemistry of portland cement

R. H. Bogue, 2nd Edition, Reinhold Publishing Corp., New York, N. Y., 1955, 790 pp., $16.50

Added material summarizing the results of progress in cement chemistry since the first edition makes this second edition one of the most authoritative, up-to-date references on the subject. Recent developments in cement chemistry and technology as a result of new exploration techniques, utilizing x-ray, electron-optical, spectroscopical and thermo-analitical methods give particular importance to the new text material on the structural characteristics of the clinker and hydrated cement compounds. Great advances in basic research have been made recently, which are particularly significant in view of the traditional lag between basic and applied research in the fields of cement and concrete technology.

Chemists and technologists engaged in cement formulation or manufacture will find in this new edition profitable new working information. It should also prove of exceptional reference value to executives, engineers, and inspectors concerned with the use of portland cement and concrete construction as well as to students of physical chemistry, ceramics, glass technology, and civil engineering.

Materials of construction

Adelbert P. Mills, Harrison W. Hayward, and Lloyd F. Rader, 6th Edition, 1955, John Wiley & Sons, Inc., New York, N. Y., 650 pp., $7.50

This excellent text and reference book has been brought up to date and expanded. Generally, individual materials of construction are described in detail in separate chapters, including a treatment on manufacture, properties, test methods, and applications of these materials. Of special interest are the sections on building stones and mineral aggregates, cementing materials, and concrete. The material on concrete aggregates is brief but comprehensive. The principal cementing materials, gypsum, lime, and portland cement, receive the treatment usual in books of this sort. In addition, there brief treatments on most of the special cementing materials. The section on concrete is more detailed than is the practice in most materials handbooks. Proportioning, physical properties, and applications, including prestressing, are discussed. Although the material on concrete in general is excellent, well supported by adequate reference to accepted research results, one unusual statement appears. It is noted that approximately 1 percent of salt in the mixing water lowers the freezing point 1 F, and further, that "beyond 10 percent, salt becomes ineffective." The book seems excellent for its primary purpose as a textbook and of considerable value as a general reference book, particularly in view of the quite extensive bibliographies attached to each section.

Cements and shrinkages (Ciments et retraits)

C. Joly, *Revue des Materiaux de Construction* (Paris), No. 471, Dec. 1954, pp. 335-341

Reviewed by Phillip L. Melville

The manufacture of ever finer portland cements is becoming a problem to the cast stone and tile industry. The fine cement gives a smooth finish, curing time must be extended, and cracking is more common. Fine cements, especially if used with a powdered pigment, prevent compaction and necessary density. Map cracking has been observed. It is hoped that a special coarse cement will be produced for the cast stone industry.

Chemical analysis and sources of air-entraining admixtures for concrete

W. J. Halstead and B. Chaiken, *Public Roads*, V. 27, No. 12, Feb. 1954, pp. 268-278

Authors' Summary

Presents results of an investigation of the practicability of applying chemical analyses as a means of identifying and controlling the uniformity of the various air-entraining admixtures used in concrete. Some such means of identification and control is needed, since the general practice at the present time is to approve a specific material for use after extended laboratory tests or actual experimental installations have shown it to be satisfactory. Subsequent purchases of the same material are then approved either

without further tests or on the basis of a limited number of short-term physical tests, but such limited tests do not necessarily give the consumer any assurance that the material purchased under the same trade name on separate occasions will be uniform in composition.

This investigation showed that air-entraining admixtures could be classified into a limited number of types, each of which contained major constituents that could be fairly easily determined in the laboratory. It is concluded that the quantitative determinations of these constituents can be used as means for adequately ascertaining the uniformity of different lots of the same materials, something which cannot be accomplished by short-term physical tests as now applied.

A qualitative scheme for the classification of unknown materials as to the type or types of compounds present was developed as a part of this investigation. The principles applied in making the quantitative determinations of the various constituents are discussed and the methods that are suitable for use in each case are given.

Influence exerted by fineness of metallurgic cements on its resistance to sulfate-containing water (Der Einfluss der Mahlfeinheit von Huttenzementen auf die Widerstandsfahigkeit gegenuber sulfathaltigem Wasser)

G. Malll, *Zement-Kalk-Gips* (Wiesbaden), V. 7, No. 11, Nov. 1954, pp. 436-438

Author's Summary

Included in investigations made with the object of ascertaining the resistance of binding agents to sulfate-containing waters, a test series was also carried out which was to show to what extent the fineness of a cement influences resisting qualities. For the test series a metallurgic cement of varying fineness was used. The test results showed that the finer the cement the higher will be its resistance to the influence of sulfate-containing water.

Prestressed concrete

Prestressed concrete (Spannbeton)

H. Moell, Berliner Union, Stuttgart, 1954, 288 pp.

Reviewed by Werner H. Gumpertz

A book dealing with the practical and construction phases of prestressed concrete work. Discussion is divided into prestressing by pre-tensioning, prestressing with tendons outside of the concrete, and prestressing by post-tensioning in general. In each chapter numerous methods of prestressing are listed, with references to patents in various countries and to other source materials. Prestressing methods in Germany, Belgium, Switzerland, England, Sweden, Italy, the Soviet Union, and United States are described. Further chapters discuss the various types of steel tendons used for prestressing, the applications of prestressed concrete, and special methods for prestressing, such as using dead loads, heat, or expanding cement to produce the prestress. An extensive list of all German patents pertinent to prestressing, and a bibliography make this book an extremely useful reference book for anybody with interest in prestressing methods.

Conveyor gantries: Prestressed concrete construction at a gasworks

Engineering (London), V. 179, No. 4642, Jan. 14, 1955, pp. 58-59

Reviewed by Aron L. Mirsky

Coal conveyor bridge 2927 ft long consists of prestressed concrete box sections, both simply supported (mostly 90-ft spans) and continuous (two spans, 108 and 78 ft). The floor of the box is a reinforced concrete slab; the sides are prestressed girders 6.5 ft deep and of varying thickness (7 in. minimum, 12 in. maximum), the ribs being continued upward to form the frame for the roof. Total depth of the box section is 10 ft 2 in.; width varies from 5 ft 8 in. to 11 ft 6 in.

Prefabricated floor and roof structures poststressed by the Gnadig-Kardos-Thoma system (in Hungarian)

I. Eber, *Magyar Epitoipar*, V. 2, No. 8, 1953, pp. 256-260

Hungarian Technical Abstracts V. 6, No. 3, 1954

The Gnädig-Kardos-Thoma (GKT) prestressing system is based on the principle of diagonal tensioning. Tensioning is effected by the diagonal stressing of steel bars. Steel bars with 130-150 kg per sq mm tensile strength are used. A 10,000 kg per sq cm initial stress is applied.

Tests are reported on 5.2 m long and 0.8 m wide elements with a 5 m inside width having two longitudinal, 5 cm wide, 27.5 cm high main ribs, and two end and three intermediate crossribs. Elements were made

of 280 kg per sq cm cube strength concrete steam cured for 8 hr at 70 to 90 C. Eight 5-mm diameter bars were used for tensioning. Cracking and failure loads were well above design loads.

Prestressed concrete slabs for railway bridges

P.S.A. Berridge, *Concrete and Constructional Engineering* (London), V. 49, No. 9, Sept. 1954, pp. 283-288

A design for railroad bridges having welded mild steel main girders calls for a deck consisting solely of prestressed precast concrete slabs. It is intended for single- or multiple-track bridges up to 60 ft long, and can be assembled quickly on the site. Connections between deck units and girders are made with high tensile steel bolts; falsework is not required.

Prestressed concrete storage building

The Engineer (London), V. 198, No. 5154, Nov. 5, 1954, p. 630

Prestressed concrete arched silo

Engineering (London), V. 178, No. 4634, Nov. 19, 1954, p. 665

Reviewed by Aron L. Mirsky

Ammonium sulfate storage building at Greenwich, England, uses three-hinged prestressed arch ribs of 86.5 ft span and 48.5 ft rise, clad with precast prestressed concrete plank.

Properties of concrete

Factors affecting resistance of portland cement concrete to scaling action of thawing agents

Albert G. Timms, *Public Roads*, V. 28, No. 7, Apr. 1955, pp. 143-157

A laboratory investigation by the Bureau of Public Roads was started in 1948 on methods of protecting concrete wearing surfaces against the action of calcium chloride. Later, a similar study was made of the effect of outdoor weather conditions on small slabs placed on the ground. It was found that resistance to scaling was affected by air content, surface treatments or coats, admixtures of oils, fly ash as a replacement for portland cement, curing methods, and by the vacuum method of placing concrete. This report is a résumé to date of investigations conducted to test materials and procedures for protecting concrete pavements. It has been found that increasing the air content in excess of 6 percent is more effective than increasing the cement content. Surface treatments of mineral oil, applied to cured and seasoned concrete, increased its resistance; oil admixtures reduced strength and gave little improvement in durability. The vacuum treatment improved the resistance of both plain and air-entrained concrete. All fly ash replacements of cement were detrimental to durability. The principal conclusion is that the resistance of concrete to calcium chloride freezing-thawing was a function of the amount of entrained air.

Shrinkage of concrete products (Retrait des produits en beton de ciment)

R. L'Hermite, *Revue des Materiaux de Construction* (Paris), No. 469, Oct. 1954, pp. 277-284

Reviewed by Phillip L. Melville

The shrinkage of hydraulic cement concrete is a function of the evaporation of mixing water, changes in weight correspond to changes in volume. Evaporation follows a law similar to the one governing cooling of a warm body. Shrinkage is proportional to the amount of capillary water evaporated, and also to the amount of hydrated cement. Shrinkage may be evaluated by the "ring" test on neat cement to measure cracking time. Shrinkage can be controlled by the use of efficient curing methods.

Corrosion of concrete and reinforcement (Corrosion du beton et des armatures)

F. Campus, *Publication Preliminaire*, London Conference 1952, International Assn. for Bridge and Structural Engineering, Zurich, Switzerland

Reviewed by Phillip L. Melville

An excellent review and discussion of concrete weathering and durability. After first pointing out that durability is relative to the expected life of the structure, the author shows that disintegration is seldom caused by internal forces (alkali reaction is extremely uncommon in Europe). Basic cause of cracking is the variation in temperature and hygrometry of the heterogenous mass. Any incipient cracking will open the way to corrosion and disintegration with progressive failure. The importance of compaction (density) and the role of entrained air are discussed. The effect of corrosive waters (sea or acid waters) is stressed. Remedies are to be found in sufficiently effective compactions of the concrete and sufficient covering of the reinforcement. Twelve references are addended.

Some tests of electro-cured concrete

A. D. Ross, *The Engineer* (London), V. 198, No. 5153, Oct. 29, 1954, pp. 582-584

Reviewed by Anos L. Mirsky

Effect of electrical curing on strength of concrete cubes, and variation of the electrical resistance of concrete with time, were investigated. Results indicated that electrical curing produced high early strength, comparable to the results of steam curing, but that prolonged heating was detrimental; further, that electrical curing seems to provide a means of accelerated strength testing offering the advantages of simplicity and almost-universal on-the-job availability.

Winter temperatures in houses with concrete floors laid on the ground

R. W. Muncey, *Australian Journal of Applied Science* (Melbourne), V. 5, No. 4, Dec. 1954, pp. 351-362

Internal temperatures for typical Australian houses in winter have been calculated alternately for concrete floors laid on the ground and in a standard timber floor over a well-ventilated crawl space. It is shown that although the thermal comfort of the two types of houses does not differ greatly, the comfort of the house with the concrete floor is slightly greater in all capital cities of Australia. The difference is comparatively small in the winter and is significant mainly during the late evening and morning. However, the summer conditions are also improved by the concrete floor and it seems that for year round thermal performance, the insulated frame, concrete floor house is the best choice.

Shrinkage of hydraulic masses, its measuring and its effects (Vom Schwinden zementgebundener Massen, seiner Messung und seinen Auswirkungen)

A. Hummel, *Zement-Kalk-Gips* (Wiesbaden), V. 7, No. 8, Aug. 1954, pp. 293-302

Reviewed by H. H. Werner

Author reviews the relationship existing between shrinkage of concrete and structure of pores, drying speed, moisture content in air, mixing ratio, cement type, and size of sample. Explains the necessity of studying, apart from the reactions produced in hardened concrete (actual shrinkage), the volume alterations in concrete which have not yet undergone the setting process (contraction), and the effects of an interrupted contraction as well as of an interrupted shrinkage.

Shrinkage phenomena depend chiefly upon the sample size and the storage climate, and measurements taken at random are of little significance for the reactions occurring in concrete in practice. Judging concrete should be based on time-shrinkage curves and a standard climate should be chosen for storage conditions to allow a proper comparison between the effects of decisive influences exerted upon the concrete. The author concludes by suggesting suitable test methods according to the above principles as a basis for further discussion.

Refractory concretes (Hormigones refractarios)

F. Arredondo, *Informes de la Construccion* (Madrid), V. 7, No. 64, Oct. 1954, Item 684-4, 4 pp.

Indicates the use of aluminous cement and refractory aggregates to make concrete for exposure to high temperatures. The properties of such concretes are discussed and mixtures suggested for various temperature ranges of exposure.

Quality concrete—Effect of mixing, placing and compacting procedures

W. L. Dawson, *Constructional Review* (Sydney), V. 27, No. 6, Oct. 1954, pp. 29-35

High quality concrete is the result of a series of processes commencing with choice of materials and mix proportioning and finishing with adequate curing. In emphasizing principles of good practice, author covers the following subjects: proportioning and effect of errors in proportioning on crushing strength; mixing, efficiency of mixers, and order and method of adding materials; transporting; placing and compacting; and curing.

Study of the tensile strength of concrete across construction joints

T. Waters, *Magazine of Concrete Research* (London), V. 6, No. 18, Dec. 1954, pp. 151-153

Author's Summary

Discusses the relative merits of various types of treatment to an existing concrete surface before fresh concrete is placed against it to form a construction joint. Results of tests of the tensile strength of joints made in various ways are given, and they show that, contrary to modern practice, old concrete should be left dry when the new concrete is placed against it. They also show

that surfaces cleaned by abrasive methods such as wire brushing and sandblasting, are slightly superior to chipped surfaces.

Results of a second series of tests are given which show that the age of the hardened concrete at the time of placing the new has little effect on the strength of the joint.

Structural research

Experimental research on plastic flow of cements (Recherches experimentales sur le fluage des ciments)

Elio Gianoreco, *Annales de l'Institut Technique du Batiment et des Travaux Publics* (Paris), July-Aug. 1954, No. 79-80

Reviewed by Phillip L. Melville

Studies of deformation under load demands a large number of test specimens and as a result research programs are unwieldy. A simple, small test specimen was selected. It consisted of an annular cylinder of neat cement. Through the hollow center was a steel rod with tightening nuts at the end faces to place the cylinder in compression. Variations in length are easily measured. Drops in compression in the specimen can quickly be corrected by tensioning the steel rod in a machine and tightening the nuts. Variations in length versus time or stress are shown graphically and might be correlated with theories such as those of Le Camus and L'Hermite.

General

Deterioration of materials

Edited by Glenn A. Greathouse and Carl J. Wessel, Reinhold Publishing Corp., New York, N. Y., 1954, 835 pp., $12

An excellent reference work to which 24 specialists contributed the results of their studies and experiences. While not limited to materials of construction, much of the information might be useful in the planning and execution of building projects where climate or the requirements of the structure pose atmospheric conditions which must be guarded against to insure reasonable durability of the building materials.

Part 1 considers some important factors in deterioration such as climate, chemical and physical agents, and biological agents. Principal components of climate — temperature, precipitation and humidity — are considered in detail. Classifications and distribution of climate are supported by data on these components. The effects of these various climates on materials are then discussed in detail.

Chemical and physical agents of deterioration reported upon are sunlight, atmospheric oxygen and ozone, atmospheric contaminants, moisture, heat and cold, abrasive materials, and wind, rain, hail, sleet, lightning, and other forces. The remainder of the book is devoted to the characteristics of various materials and assemblies, and their preservation.

Wood forms and wire ties—V. 2—The forming of shells, walls, slabs, and stairs (in French)

C. Kupfer, Editions Eyrolles, Paris, 1955, 128 pp., 950 F.

V. 1 (see "Current Reviews," ACI Journal, June 1954, *Proc.* V. 50, p. 916) treated the forming of foundations, columns, and beams. Present volume covers in similar manner the forming of other structural members. Treatment is comprehensive and well illustrated but author confines himself, in conformity with his title, to wire ties and omits mention of manufactured metal ties almost universally used in best practice today. Wood forms described are almost all of dimensioned lumber with little attention to the important and growing use of plywood for forming. Description of round wood posts for shores and means of connecting them together is a reflection of a practice little used in the United States where adjustable shores would be preferred.

The book is of historical interest and of reference value to the builder of forms for such special purposes as quarter-turn stairways where standardized materials may not be applicable.

New technique for repairing spalled joints

Public Works, V. 85, No. 11, Nov. 1954, pp. 68-69

The spalled joints were repaired using an unusual technique. The spalled portions were removed, the area to be patched was thoroughly cleaned, and filled with a dry tamped portland cement topping.

In removing the damaged area at the joints, a concrete saw was used to make a vertical cut, 1½ to 2 in. deep, approximately 6 in. in back of the contraction joint, and extending transversely for the length of the damaged section. Success of this type of patching depends to a large degree on the mixture.

Proportions were 1 to 5, the aggregate consisted of 60 percent sand and 40 percent coarse pea gravel. Water was added to give a dry consistency and an air-entraining agent and 2 percent of calcium chloride were added to the mixing water for high-early strength and durability.

These patches were inspected after being subjected to 15 months of heavy traffic and exposure to one winter of freezing and thawing. The excellent condition of the patches at that time demonstrates that construction defects of this type can be corrected, using concrete.

Geology in engineering

JOHN R. SCHULTZ and ARTHUR B. CLEAVES, John Wiley & Sons, Inc., New York, N. Y., 1955, 590 pp., $8.75

Designed both as a text and practical reference, this book integrates geology and engineering. It gives a systematic account of geologic principles and methods for the benefit of the engineer. It places considerable emphasis on soils and application of geology to soil mechanics. The treatment is analytical rather than descriptive with more attention being paid to principles than to case histories, though these are represented. The book shows considerable effort to achieve a balance between principles of geology and engineering applications. It is well illustrated with clear photographs and diagrams and includes full bibliographical references. There are detailed chapters on geologic structure, historical geology, geologic maps and sections, aerial photographic interpretation of soils, and subsurface exploration. The book ends with separate chapters on engineering applications to dams, tunnels, paving, and concrete aggregates. Used as a text it should make the study of geology more interesting to engineering students, and it may very well be used as the basis for required courses in geology for civil engineering students.

Chemical plant lined with rubber

British Constructional Engineer (London), V. 5, No. 2, Sept. 1954, pp. 47, 49

Plant built in England processes bromine out of sea water. In this process, part of which takes place in concrete cells, the sea water is acidulated and then treated with gaseous chlorine, whereby bromine is liberated and removed by counter currents of air through the cells. Treatment with sulfur dioxide causes formation of fine mists of hydrobromic and sulfuric acids, which are filtered out of the air and the resulting liquor treated for liberation of the bromine itself.

Provision for acidulating the sea water and handling the various acid mists necessitated thorough protection of the concrete, steel, and other materials. This was achieved throughout the plant by rubber and ebonite linings and coverings. The concrete cells (blowing-out tower) were lined with unvulcanized rubber sheet and cured in place by hot air. The maximum permissible temperature was 90 F, since the concrete was not allowed to rise above this temperature. During vulcanization thermocouples embedded in the cell walls were used for temperature recording and control. The surface covered was more than 75,000 sq ft.

The cellular slab—greater strength at lower cost

DOMENICO PARMA, *Consulting Engineer*, V. 4, No. 6, Dec. 1954, pp. 48-51

Reviewed by ARON L. MIRSKY

Lower halves of concrete "pan blocks" are placed on forms, reinforcing steel placed in spaces between blocks, and top halves of the pan blocks placed, after which concrete is placed between the blocks to form a "reticular cellular slab." Advantages are stated to include good resistance to earthquakes or wind, ability to carry heavy concentrated loads, good sound absorption, and ease of finishing ceilings. Calculations are similar to those for conventional flat slabs. Tests indicate deflection corresponds to that of monolithic beams of double-T section. System has been extensively used in Colombia since 1948.

Concrete testing with gamma rays (Essais par rayons gamma sur les betons)

J. P. FACKLER, *Cahiers du Centre Scientifique et Technique du Batiment* (Paris), No. 21, Dec. 1953

Reviewed by PHILLIP L. MELVILLE

A basic theoretical and applied investigation of concrete by gamma rays from a radio-cobalt source. Variations in density were determined with 1 percent accuracy in concrete mixes. Lead filled wood "aggregates" were used to study the effect of

vibration by means of gamma graphs. A number of defects could also be located by the method. Safety measures needed in handling radio-cobalt 60 (or less useful iridium) are carefully stressed. Reference is made to the report of the British Road Research Laboratory in *The Engineer*.

Radiators made from concrete (in Hungarian)
A. Macskasy, *Epuletgepeszet*, V. 2, No. 7-8, 1953, pp. 134-140
Hungarian Technical Abstracts V. 6, No. 3, 1954

Deals with the causes which have prevented the practical use of radiators made from concrete. It is considered incorrect to model concrete radiators after the ordinary cast iron or steel plate radiators. The most practical shape for radiators made from concrete is that of radiating slabs. Advantages are the following: production is simple; the weight may be substantially reduced; increased utilization of radiation; most favorable conditions of radiation occur near the floor of the room; it permits high radiator temperatures.

The following formula is established for the determination of the heating effect of the pipe coils built into the concrete slabs:

$$q_p = 2\delta_o \sqrt{i\lambda_i (a_i + a_e)}\, th \frac{ml}{2}$$

where q_p = heat transferred by running meter of piping, δ_o = excess temperature of the heating agent compared to the temperature of the room, i = thickness of the concrete radiator, λ_i = coefficient of thermal conductivity, a_i = coefficient of heat transfer at one side of the slab, a_e = same on the other side, $m = \sqrt{\frac{a_i + a_e}{i\lambda_i}}$ characterizing the thermal properties of the slab, l = distance between the heating pipe coils.

The principal viewpoints to be considered in designing radiators made from concrete are represented by this formula. Heat losses due to the heating of the wall surface behind the radiator are computed subsequently. The heat radiated by a 6 cm thick radiator occupying 1 sq m of space and the most economical distance between the heating pipe coils are determined by taking into account the changes in the coefficient of the heat transfer as a function of the surface temperature in reference to the former. Thus the utilizable heat radiated by both sides of a concrete radiator taking up 1 sq m of space is 1300 kcal per hr. In this case each running meter of heating pipe transfers 178 kcal of heat per hr which far surpasses that attained by any other type of radiator.

Economical specifications for concrete
The Engineer (London), V. 198, No. 5159, Dec. 10, 1954, p. 813
Reviewed by Aron L. Mirsky

Since 1952 the British Ministry of Works has been using performance-type specifications for concrete: the minimum strength and any special requirements such as water tightness, etc., being specified and the contractor being permitted to design the mix to suit the available aggregates. Results have been highly successful. Also reported briefly here are results of investigations by the ministry on techniques designed to reduce costs still further: a power float, a vibrating roller for slabs on ground, and early stripping of forms.

Standards for the construction and protection of piers and wharves
NFPA Standard No. 87, National Fire Protection Assn., Boston, Mass., May 1954, 24 pp., $0.35

A standard on the design and construction of piers and wharves aimed at securing adequate fire safety. Both substructures and superstructures are covered for fire-resistive, noncombustible, and combustible construction.

Concrete on the railways
Concrete Assn. of India, Bombay, 1954, 40 pp.

This pictorial brochure gives some idea of the many uses of concrete by railroads in India. A few illustrations are included of concrete construction by European and American railroads.

Horizontal cracks in concrete floor slabs. Assessing cracking below granolithic surfaces
Engineering (London), V. 178, No. 4632, Nov. 5, 1954, p. 601
Reviewed by Aron L. Mirsky

Describes apparatus developed for determination of depth and thickness of horizontal cracks in slabs with granolithic topping; extent of cracks being determined as usual by sounding.

PERIODICALS REGULARLY SCANNED FOR REVIEWS

The many requests for information concerning the source material of reviews appearing in the "Current Reviews" section indicate the desirability of publishing a list of those periodicals, bulletins, and proceedings being *regularly scanned* for articles dealing with the field of concrete. If all miscellaneous magazines, bulletins, reports, and books were included (which are reviewed only as they come to the attention of the voluntary reviewers or editors) the following list would cover well over 200 periodicals and/or publishing, educational, and research organizations. The following list includes publishers' addresses for convenience should correspondence with them be desired, since copies of articles or books reviewed in the JOURNAL are not available through ACI. In most cases they can be obtained directly from the original publishers.

American Ceramic Society Bulletin	American Ceramic Society, 2525 North High St., Columbus 2, Ohio
AIA—The Monthly Bulletin	Michigan Society of Architects, 120 Madison Ave., Detroit 26, Mich.
AREA Bulletin	American Railway Engineering Assn., 59 East Van Buren St., Chicago, Ill.
ASTM Bulletin	American Society for Testing Materials, 1916 Race St., Philadelphia 3, Pa.
Annales de L'Institut Technique du Batiment et des Travaux Publics	19 Rue La Perouse, Paris XVI^e, France
Annales des Ponts et Chaussees	Commission des Annales a l'Ecole National des Ponts et Chaussees, 28 Rue des Saints-Pères, Paris, France
Applied Mechanics Reviews	American Society of Mechanical Engineers, 29 West 39th St., New York 18, N. Y.
Architectural Forum	Time & Life Bldg., 9 Rockefeller Plaza, New York 20, N. Y.
Architectural Record	119 West 40th St., New York 18, N. Y.
Asamblea General	Instituto Tecnica de la Construccion y del Cemento, Costillares (Chamartin de la Rosa) Apartado No. 2, Madrid, Spain
Australian Journal of Applied Science	Commonwealth Scientific and Industrial Research Organization, 314 Albert St., East Melbourne, C 2, Victoria, Australia
Australian Journal of Scientific Research, Series A: Physical Sciences	Commonwealth Scientific and Industrial Research Organization, 314 Albert St., East Melbourne, C 2, Victoria, Australia
Bauingenieur, Der	Reichpietschufer 20, Berlin, W 35, Germany
Bauzeitung, Die	Deutscher Fachzeitschriften- und Fachbuch-Verlag GmbH, Hospital strasse 12, Stuttgart N, Germany
Betong	Odengatan 41, Stockholm, Sweden
Beton og Jernbeton	Institution of Danish Civil Engineers, 29 Vester Farimagsgade, Copenhagen V, Denmark
Betonstein-Zeitung	Frankfurter Strasse 5, Wiesbaden, Frankfurt Am Main, Germany
Beton-Teknik	Christians Brygge 28, Copenhagen V, Denmark
Beton- und Stahlbetonbau	Hohenzollerndamm 169, Berlin-Wilmersdorf, Germany

British Constructional Engineer, The	32 Southwark Bridge Road, London, SE1, England
Building Research Reports, Danish National Institute of Building Research	20 Borgergade, Copenhagen K, Denmark
Building Research Station Digest	Building Research Station, Garston, Watford Herts, England
Bulletin, Centre d'Etudes et de Recherches de L'Industrie des Liants Hydrauliques	197 Boulevard Saint-Germain, Paris VII^{e}, France
Bulletin de la Reunion des Laboratories d'Essais et de Recherches sur les Materiaux et les Constructions	12 Rue Brancion, Paris XV^{eme}, France
Bulletin, Faculty of Engineering, Cairo University	Cairo University, Giza, Egypt
Bulletin, Highway Research Board	Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.
Bulletin, Institution of Civil Engineers in Ireland	35 Dawson St., Dublin, Ireland
Bulletin, Instituto Tecnico de la Construccion y del Cemento	Costillares (Chamartin de la Rosa) Apartado No. 2, Madrid, Spain
Bulletin, Iowa Engineering Experiment Station	Iowa Engineering Experiment Station, Iowa State College, Ames, Iowa
Bulletin Technique de la Suisse Romande	Case Postale Riponne, 21 Lausanne, Switzerland
Bulletin, University of Illinois Engineering Experiment Station	University of Illinois Engineering Experiment Station, 106 Engineering Hall, Urbana, Ill.
Cahiers du Centre Scientifique et Technique du Batiment	4 Ave. du Recteur-Poincaré, Paris 16^{e}, France
Cement	Sarphatistraat 54, Amsterdam C, Netherlands
Cement and Lime Manufacture	14 Dartmouth St., Westminster, SW 1, London, England
Cement och Betong	Kolga-huset, Postgiro 1808, Malmö, Sweden
Cemento, Il	1 Viale F. Testi, Milan, Italy
Cemento Portland	Calle San Martin 1137, Buenos Aires, Argentina
Ceramic Abstracts	(see *Journal of the American Ceramic Society*)
Civil Engineer Corps Bulletin	Bureau of Yards and Docks, Department of the Navy, Washington 25, D. C.
Civil Engineering	33 West 39th St., New York 18, N. Y.
Civil Engineering and Public Works Review	8 Buckingham St., Strand, London, WC 2, England
Commonwealth Engineer, The	349 Collins St., Melbourne, C 1, Australia
Concrete	400 W. Madison St., Chicago 6, Ill.
Concrete and Constructional Engineering	14 Dartmouth St., Westminster, SW 1, London, England
Construction Methods and Equipment	330 West 42nd St., New York 18, N. Y.
Constructional Review	14 Spring St., Sydney, N.S.W., Australia
Consulting Engineer	420 Main St., St. Joseph, Mich.
Contractors and Engineers	470 Fourth Ave., New York 16, N. Y.
Crushed Stone Journal, The	1415 Elliot Place, NW, Washington 7, D. C.
Engineer, The	28 Essex St., Strand, London, WC 2, England
Engineering	35 & 36 Bedford St., Strand, London WC 2, England

Engineering Journal, The	2050 Mansfield St., Montreal 2, Quebec, Canada
Engineering News-Record	330 West 42nd St., New York 18, N. Y.
Genie Civil, Le	5 Rue Jules Lefebvre, Paris 9e, France
Geotechnique	Institution of Civil Engineers, Great George St., London, SW 1, England
Giornale del Genio Civile	Via Nomentana 2, Rome, Italy
Gradbeni Vestnik	Erjavceva 11a, Ljubljana, Jugoslavia
Handlingar (Proceedings), *Swedish Cement and Concrete Research Institute*	Svenska Forskningsinstitutet for Cement och Betong vid Kungl. Tekniska Hogskolan I, Stockholm, Sweden
Highway Research Abstracts	Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.
Housing Research Papers, HHFA	Housing and Home Finance Agency, Division of Housing Research, Washington 25, D. C.
Hungarian Technical Abstracts	Hungarian State Technical Library, P.O. Box 12, Budapest 8, Hungary
Indian Builder, The	Janmabhoomi Chambers, Fort St., Bombay 1, India
Indian Concrete Journal, The	1 Queen's Road, Bombay, India
Industrial and Engineering Chemistry	American Chemical Society, 1155 Sixteenth St., NW, Washington 6, D. C.
Informes de la Construccion	Instituto Tecnico de la Construccion y del Cemento, Costillares (Chamartin de la Rosa) Apartado No. 2, Madrid, Spain
Ingenieria Civil	Plaza de la Republica No. 55-602, Apartado Postal 10423, Mexico 1, D. F., Mexico
Ingenioren	Copenhagen, Denmark
Journal of Boston Society of Civil Engineers	715 Tremont Temple, Boston, Mass.
Journal of the American Ceramic Society and Ceramic Abstracts	2525 N. High St., Columbus 2, Ohio
Journal of the Institution of Engineers, Australia	Gloucester & Essex Sts., Sydney, N.S.W., Australia
Journal of the Japan Society of Civil Engineers	No. 4, 2-Chome, Ohte-Machi, Chiyoda-Ku, Tokyo, Japan
Journal of Research	National Bureau of Standards, Washington 25, D. C.
Magazine of Concrete Research	52 Grosvenor Gardens, London, SW 1, England
Meddelanden (Bulletins), *Statens Provningsanstalt*	Statens Provningsanstalt, Stockholm, Sweden
Memoires de la Societe des Ingenieurs Civils de France	Hotel de la Societe, 19 Rue Blanche, Paris 9, France
Military Engineer, The	Society of American Military Engineers, Mills Bldg., Pennsylvania Ave. at 17th St., NW, Washington 6, D. C.
Nase Gradjevinarstvo	Assn. of Societies of Civil Engineers and Technicians of Yugoslavia, Kneza Milosa 7, Belgrade, Yugoslavia

Oesterreichische Bauzeitschrift	Oesterreichischer Betonverein, Strohgasse 21 A, Vienna III, Austria
Ontario Hydro Research News	Research Division, Hydro-Electric Power Commission of Ontario, 620 University Ave., Toronto 2, Ont., Canada
Pit and Quarry	431 S. Dearborn St., Chicago 5, Ill.
Polish Technical Abstracts	Centralny Instytut Dokumentacji Naukowo-Technicznej, Aleja Niepodleglosci 188, Warsaw, Poland
Polytechnisch Tijdschrift	28 Balistraat, The Hague, Netherlands
Precontrainte	21 Rue Newton, Brussels 4, Belgium
Prefabrication	4 Vernon Place, London, WC 1, England
Proceedings, ASCE	American Society of Civil Engineers, 33 West 39th St., New York 18, N. Y.
Proceedings, ASTM	American Society for Testing Materials, 1916 Race St., Philadelphia 3, Pa.
Proceedings, Deutscher Ausschuss fuer Stahlbeton	Bundesallee 215/218, Room 302, Berlin, W 15, Germany
Proceedings, Deutscher Beton-Verein	Deutscher Beton-Verein E.V., Bahnhofstrasse 61, Postfach 627, Wiesbaden, Germany
Proceedings, Highway Research Board	2101 Constitution Ave., Washington 25, D. C.
Proceedings, Institution of Civil Engineers	Great George St., Westminster, London, SW 1, England
Publicaciones Tecnicas del Instituto del Cemento Portland Argentino	Instituto del Cemento Portland Argentino, Calle San Martin 1137, Buenos Aires, Argentina
Publications, International Assn. of Bridge and Structural Engineering	Verlag Ag. Gebr. Leeman & Co., Stockerstrasse 64, Zurich 39, Switzerland
Public Roads	Bureau of Public Roads, U. S. Dept. of Commerce, Washington 25, D. C.
Rakennusinsinoori	Ratak 9, Helsinski, Finland
Reinforced Concrete Review, The	94-98 Petty France, London, SW 1, England
Revista de Engenharia Mackenzie	Rua 7 de Abril, 230, Sao Paulo, Brazil
Revista de Obras Sanitarias de la Nacion	Charcas 1840, Buenos Aires, Argentina
Revista del Colegio de Ingenieros de Venezuela	Parque Los Caobos, Apartado 2.006, Caracas, Venezuela
Revue des Materiaux de Construction	19 Rue Lafayette, Paris 9, France
Roads and Streets	24 W. Maple St., Chicago 10, Ill.
Rock Products	309 W. Jackson Blvd., Chicago 6, Ill.
Schweizerische Bauzeitung	Schweizerischer Ingenieur-und Architekten-Verein, Postfach, Zurich 39, Switzerland
Stavebni Prumysl	Spalena 51, Prague II, Czechoslavakia
Structural Engineer, The	2 Bream Bldgs., London, EC 4, England
Surveyor and Municipal and County Engineer, The	Carisle House, 8 Southampton Row, London, WC 1, England
Technical Information Digest for Municipal and Airport Engineers and Officials	American Road Builders' Assn., 1319 F St., NW, Washington 4, D. C.
Technical Memorandum, Waterways Experiment Station	Waterways Experiment Station, Vicksburg, Miss.

Tecnica	Revista de la Facultad de Ciencias Exactas y Tecnologia, Universidad Nacional de Tucuman, Argentina
Technique Moderne-Construction, La	92 Rue Bonaparte, Paris 6, France
Teknisk Tidskrift	Box 841, Stockholm 2, Sweden
Teknisk Ukeblad	Oslo, Norway
Transactions, Institution of Civil Engineers of Ireland	35 Dawson St., Dublin, Ireland
Transactions, Japan Society of Civil Engineers	No. 4, 2-Chome, Ohte-Machi, Chiyoda-Ku, Tokyo, Japan
Travaux	6 Ave. Pierre, 1st de Serbie, Paris 16, France
Trend in Engineering at the University of Washington	127 More Hall, University of Washington, Seattle 5, Wash.
Western Construction	609 Mission St., San Francisco 5, Calif.
World Construction	24 W. Maple St., Chicago 10, Ill.
Zeitschrift des VDI	Deutscher Ingenieur-Verlag GmbH, Ingenieurhaus, Prinz-Georg-Strasse 77, Duesseldorf, Germany
Zement-Kalk-Gips	Frankfurterstrasse 5, Wiesbaden, Germany

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